REGIONAL MUNICIPALITY OF HALTON

Halton Hills (Georgetown) Urban Area Review Sewerage & Sewage Treatment Study

Municipal Planning Consultants
in Association with
Gore & Storrie Limited

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LETTER OF TRANSMITTAL

24 February 1982 603.01

Chairman and Members
Halton Hills Urban Area Review Steering Group
The Regional Municipality of Halton
1151 Bronte Road
Oakville, Ontario

Gentlemen:

Re: Stage I
Halton Hills (Georgetown) Urban Area Review
Your File No. 3606.1

We take pleasure in submitting herewith our report on the Halton Hills (Georgetown) Urban Area Review Sewerage and Sewage Treatment Systems Review.

This study commenced in October 1980, and involved extensive collection monitoring, Water Pollution Control Plant optimization and monitoring, and stream monitoring in Rogers Creek.

Based on our investigation of the sewerage system we observed general infiltration over the entire serviced area however not of a magnitude greater than experienced elsewhere in the Region. We are of the opinion that extensive improvements to the existing system will achieve little benefit at a great cost and are not economically warranted. New sewers will be constructed to current design standards which reduce the potential for excessive inflow and infiltration. We believe the projected flows indicated in this report are conservative and attainable.

The existing Water Pollution Control Plant performed well throughout the study period and under normal conditions met Ministry of the Environment Provincial Water Quality Objectives.

In order to ascertain the limiting parameters in the treatment process testing procedures were established to overload both organically and hydraulically one section of the plant. The findings of this study have permitted us to define more accurately design parameters for an expanded facility to treat the present committed treatment flow of 13,600 m^3/day as well as the future ultimate projected flow of 22,730 m^3/day.

We are confident the treatment facility can be expanded to service a population of 40,000 persons and meet the Ministry of the Environment requirements as set out in their letter of December 30, 1981 to Gore & Storrie Limited.

We trust that you find this report complete and conclusive. We wish to acknowledge the co-operation and assistance of the Steering Group, the Region of Halton Planning staff and Public Works Department staff and the Town of Halton Hills Planning and Engineering Departments staff and the Ontario Ministry of the Environment without whose help our task would have been more difficult.

All of which is respectfully submitted

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CHAPTER 1

SUMMARY

CHAPTER 1

SUMMARY

This report addresses the technical aspects of the Halton Hills (Georgetown), Urban Area Review Sewerage and Sewage Treatment Systems Review. The study commenced in October of 1980 and the field portion was completed in October of 1981. During this period the study examined the following:

1. A review of previous reports prepared on the Georgetown urban area with respect to sewerage and sewage treatment.
2. An examination of the sewerage system and its performance under various conditions of average flow, diurnal peaks and the peak flows as a result of wet weather conditions.
3. An examination of the performance of the water pollution control plant under all of the above conditions, with specific attention to the ability of the plant to perform under winter conditions while approximately half the plant was shutdown.
4. An evaluation of the existing pollutant background levels in Rogers Creek.
5. An examination of the impact of the plant effluent on Rogers Creek.

The study identified a level of infiltration and inflow across the sewage system greater than desirable but not excessive. It is believed that with new construction, the inflow and infiltration levels may be reduced by approximately 50%, resulting in equivalent per capita flows of approximately 477 litres/capita/day (105 gal/capita/day).

The study of the water pollution control plant determined that the plant routinely turns out an effluent of high quality meeting the current requirements of the Ministry of the Environment. The daily average flow is about 10,000 m^3/day (2.2 MIGD) with peaks normally not exceeding

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22,700 m^3/day (5 MIGD). We estimate that when the average flows exceed 11,400 m^3/day (2.5 MIGD) the quality of the effluent from the water pollution control plant will begin to deteriorate. In order to meet the objectives of the Ministry of the Environment set out in their letter of December 30, 1981 modifications to the water pollution control plant are required to meet the present 30,000 population limit.

Based upon the plant survey and the stream study we have prepared a number of recommendations for future action which we believe will permit growth in the Georgetown area to a population of 40,000 persons.

The modifications required to meet the 30,000 person limit may be marginally increased to accomodate a population of 40,000 persons while still maintaining an effluent meeting the ministry's objectives. These modifications would set the plant capacity at 22,730 m^3/day at an estimated 1982 cost of $7,130,000.

CHAPTER 2

INTRODUCTION

CHAPTER 2

INTRODUCTION

2.1 General

At present the urban development of Georgetown in the Town of Halton Hills is limited in its growth potential to 30,000 persons population. This limitation is due to a restriction on the water pollution control plant flow to 13,700 m^3/day (3 MIGD). A number of reports have been prepared in the past outlining problems with infiltration and inflow into the sewage system further cutting down on the ability of the urban area to accept new growth. Because of the restriction and because of the desire by a number of the developers in the area to develop their lands, the Region of Halton has inititated the following study to determine:

A.) The increased equivalent population if any which can be served at the existing water pollution control plant, and

B.) Further increases in equivalent population which can be permitted using additional wastewater treatment techniques acceptable to the Ministry of the Environment.

Muncipal Planning Consultants Limited in association with Gore & Storrie Limited were engaged in September 1980 to carry out a technical review of the sewage system, the water pollution control plant operation and the effects of the water pollution control plant effluent on the receiving stream, Rogers Creek.

The study commenced in October 1980 and had a duration of approximately one year during which the sewage system operation was monitored to determine the average dry weather flow, the diurnal flow variations and points of excessive inflow and infiltration into the sewers. Previous reports on the sewage collection system and water pollution control plant

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were reviewed and considered in the preparation of our study program and during the program itself.

The water pollution control plant monitoring program was broken into four finite sections with monitoring being carried out on a weekly basis to determine the waste strengths and characteristics in various elements of the treatment process and in particular the effluent.

The flows in Rogers Creek were monitored upstream, at the point of effluent discharge, 50 metres downstream and at the confluence of Rogers Creek and the Credit River for a number of criteria outlined in more detail later in the report. Four separate stream monitoring periods were proposed by the Ministry of the Environment, one for each season. Samples were taken, stream flows measured and the samples transported to the Ministry of the Environment laboratories in Toronto for analysis.

Meetings were held throughout the study period for direction and to update the Halton Hills (Georgetown) Urban Area Review Steering Group of the current status of the study. Additional meetings were held from time to time with the Ministry of the Environment for their acceptance of the progress of the study and the projected outcome.

The technical portion of the study was completed in October of 1981 with the analysis being completed from October through December and the preparation of the report by mid January 1982. We believe that the plant and the system have been throughly analyzed.

CHAPTER 3

STUDY PROCEDURES

CHAPTER 3

STUDY PROCEDURES

3.1 General

The study was separated into 5 basic tasks:

1. Meetings
2. Background Review
3. Sewage System Survey including pumping stations
4. Water Pollution Control Plant Monitoring
5. Stream Monitoring

This Chapter outlines the procedures set out for each of these tasks.

3.2 Meetings

Regular meetings were held both at the request of the Steering Group and at the request of the consultant to report on the progress of the study and to request direction and assistance prior to proceeding with the next steps in the study.

An initial meeting was held with the Ministry of the Environment to discuss the work program and to determine to what the extent participation from the Ministry of the Environment would be available with respect to laboratory testing. It appeared at an early date that full participation by the Ministry laboratory was not possible and would be limited to those tests which we could not carry out in our own laboratories. We therefore proceeded with an in-house laboratory analysis program with the exception of specific analyses required in the stream assimilation work.

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Following the meeting with the Ministry of the Environment a meeting was held with the Steering Group to ensure that the Steering Group was satisfied with the work program as proposed.

A meeting was held in June of 1981 with representatives from the Town, the Region, the Ministry of the Environment, Municipal Planning Consultants and Gore & Storrie to discuss the status of the study to date and to obtain an appreciation of the Ministry's reaction to the findings of the study to date.

The Ministry indicated at that meeting that tney were satisfied with the progress on the study to date and that based upon the present per capita flow of 454 l/capita (100 gal/capita) a population of 40,000 persons could be considered, subject to the findings in the final report. Based upon this early indication by the Ministry, the 1st phase of the planning study and the remainder of the sewerage and sewage treatment continued.

In November the Steering Group requested a meeting with the Ministry of the Environment to obtain their position prior to the commencement of the phase 2 portion of the planning study. A meeting was held December 18, 1981 with representatives of the Ministry, the Region, the Town, Municipal Planning Consultants and Gore & Storrie Limited at which time considerable discussion was held with respect to the effects of the sewage effluent on Rogers Creek, the contaminant background levels in Rogers Creek and improvements which may take place in the Creek as a result of:

a) Changes at the Acton Water Pollution Control Plant

b) An improvement in the effluent quality of the Georgetown water pollution control plant by additional treatment allowing an increase in flow.

The Ministry indicated that if it could be demonstrated that it is possible to expand and improve the existing water pollution control plant to meet the Ministry guidelines with respect to toxicity in the receiving stream then a plant expansion could be considered and the population of 40,000 population proposed at the June meeting would still be acceptable.

3.3 Background Review

The following have been reviewed in detail and referred to during our investigations of the system and water pollution control plant.

1. Report and Plans by Proctor and Redfern Limited, dated April 1977 and February 1979.
2. Reports by Phillips Planning and Engineering Limited
3. Study on the Assessment of Sewage Plant Sludge Disposal for the Region by Simcoe Engineering.
4. The Ministry of the Environment Draft Report on the Assessment of the Present Plant Discharge and Its Effect on Rogers Creek during 1979.

In addition to the above, various drawings, plans, reports and interviews with Regional and Town staff were carried out, all of which have been taken into consideration during the preparation and implementation of the study procedures and the preparation of this report.

3.4 Collection System Monitoring

The present Georgetown sanitary sewerage system consists of approximately 72 kilometres of sanitary sewers ranging in size from 200 mm to 750 mm diameter, 3 sewage pumping station's within the collection system, a main sewage pumping station at the plant site, and a treatment plant with a design capacity of 13,600 m^3 per day (3 mgd).

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The study area was divided into sewage collection zones with each zone being monitored regularly over a series of months. This monitoring program provided dry weather flow data for the various time periods throughout the day. These measurements are summarized in Appendix 'A' and the collection zones are shown on Appendix 'A' of Appendix 'D'.

Once sufficient dry weather flow data had been obtained, monitoring was limited to major storm events and high flow conditions.

Two sewers discharge to the plant joining in a junction chamber. A flow meter was installed in this junction chamber, (manhole No. 1), to provide a record of each high flow condition. The meter also acted as a backup unit to check whether or not this inlet trunk sewer was a source of higher flows. The meter's recording chart was changed weekly and compared to plant meter records over the same period.

Included in the flow monitoring program were spot checks of some residential roof drains. In those tested, dye was placed in the house roof drains during a storm event to determine whether or not the drains were connected to the storm sewer system or the sanitary sewer system.

3.5 Water Pollution Control Plant Monitoring

The Georgetown water pollution control plant is composed of two separate treatment plants, A and B, each with its own primary and secondary treatment elements.

The field work for the water pollution control plant monitoring program was commenced in September 1980. Portable sampling equipment was provided by Gore & Storrie throughout the study because there was no sampling equipment in plant. A permanent sampler was installed on the chlorine contact tank effluent by the Region but was not operational through the sampling program.

The only permanent flow meter installed at the commencement of the study was on the chlorine contact chamber influent. No flow information on return activated sludge or waste activated sludge was available from the water pollution control plant records or meters. To resolve this deficiency a temporary waste activated sludge pump and metering system was provided and installed in plant B. This system was placed into service in February 1981.

Plant B was selected as the test unit for the following reasons. It had:

1. A smaller aeration tank volume.
2. Larger secondary clarifiers.
3. A more reliable return activated sludge system.
4. A completely mixed activated sludge system as opposed to the plug flow system operating in plant A.

Sampling equipment was provided to obtain samples of all the relevant flow streams in plant B. 24-hour composite samples taken at the following locations:

1. Grit removal system effluent
2. Primary clarifier effluent
3. Secondary clarifier effluent
4. Chlorine contact chamber effluent.

Grab samples of mixed liquor and return activated sludge were analyzed as required.

Plant operating records were obtained and checked to provide the following:

1. Total sewage flow for each day
2. Maximum sewage flow rate for the day.
3. Amount of phosphorus removal chemical applied each day.
4. Amount of waste activated sludge produced each day.

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To study the operation of the plant under various loadings the following operating conditions where selected:

CONDITION	PLANT A	PLANT B	TIME PERIOD
1	In Service 4,300m3/d	In Service 4,300m3/d	October 9 to February 1981
2	Out of Service	In Service 9,000m3/d	February 17 to June 19
3	In Service 3,000m3/d	In Service 5,600m3/d	June 24 to August 20
4	Out of Service	In Service 10,000m3/d	August 20 to August 21

A summary table of operation of Plant B is presented opposite page 4-4.

Plant operations were adjusted to optimize the performance throughout the study. For short periods plant loadings were artificially increased to examine the effect on the treatment process. Specifically additional Ferric chloride was added to the phosphorus removal process to examine the ability of the plant to reduce phophorus levels to less than 0.3 mg/l.

3.6 Rogers Creek Monitoring Program

Water quality sampling of Rogers Creek was done to determine its water quality for the designated creek flow 7Q20 (low 7 day average flow expected every 20 years) of 0.17 m^3/s in the winter and 0.144 m^3/s in the summer for the present WPCP flows. This was done also to determine the expected creek water quality for increased flows from the WPCP up to 0.26 m^3/s (5 MGD).

Rogers Creek was sampled 100 m upstream of the WPCP discharge, at the discharge, 50 m downstream of the discharge and at Highway 7 (see Figure 1). Samples were taken from the centre of the Creek during 1981 on January 23 and 30th, February 5 and 12th, March 12th, April 2, 9, 14 and 23, July 23 and 30th, August 5 and 13th, September 30th and October 7, 13, and 21.

The following parameters were measured:

Creek Physical Parameters

Water depth, width, temperature, velocity, total dissolved and suspended solids, pH, chlorite, and conductivity.

Creek Chemical Characteristics

Nutrient concentrations (total and dissolved phosphorus, ammonia, nitrate, nitrite and kjeldahl nitrogen), dissolved oxygen, carbonaceous biological oxygen demand, biological demand (nitrogen inhibited), total residual chlorine and dissolved oxygen.

Creek Biological Characteristics

Total and fecal coliforms, fecal streptococci.

Standard Methods were used for the physical and chemical tests. The results are presented in Tables 1 to 6 in Appendix 'C'.

Creek flow data were obtained from river crossection measurements at 100 m upstream of the discharge, 50 m downstream and at Highway 7 (see Figures 2 and 3 and Table 7 in Appendix 'C').

CHAPTER 4

DISCUSSION

CHAPTER 4

DISCUSSION

4.1 Review of Earlier Reports

As described in Chapter 3, the study included review of previous reports on the sewage system, pumping stations and the water pollution control plant. It also included detailed field examinations and many discussions with Regional personnel to determine what action had been taken on the recommendations of the reports.

A letter was sent by Gore & Storrie to the Region in January of 1981 recommending remedial action be taken in a number of locations in the sewer system. This is the same as set out on the tables in Appendix 'A'.

Some of these areas had been referred to in the previous reports by Proctor and Redfern Limited and some were discovered during our early investigations into the sewer system.

We carried out a review of the Sludge Management Study by Simcoe Engineering for Halton Region with special attention to the comments on the Georgetown water pollution control plant and its operation. A major recommendation of this study was that the existing digester system should be either substantially overhauled or completely replaced at an early date so that the sludge from this plant could be disposed of on agricultural land as has been done in the past.

We concur with the recommendations of the Simcoe Engineering report but would go one step further and recommend that the existing digester system be abandoned and be replaced utilizing gas mixing and solids recirculation.

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4.2 Sewage System Survey

4.2.1 Sewage System Monitoring Program

Infiltration, although not severe, is widespread throughout the sewage collection system. The terms of reference for the study area outlined specific locations within the collection system that required investigation to determine whether or not infiltration of the system could be reduced. These are detailed in Appendix 'A' with recommendations.

Monitoring the individual manholes in each zone during periods of dry weather flow provided good representative data for various time periods throughout the day. Regular planned visits to the study area were discontinued after collection of dry weather flow information in favour of reacting to major storm events and high flow conditions.

4.2.2 Pumping Station Review

The pumping stations were reviewed for their mechanical fitness and are discussed in detail with recommendations in Appendix 'A'. Changes are required at each of these stations to improve their operation.

4.2.3 Sewage Flow Generation

The water sales records and sewage flow generation records for Georgetown for the years 1975 through 1980 were reviewed. However, the sewage flow generation rates prior to 1978 were found questionable due to the poor accuracy of the flow measurement equipment at the water pollution control plant.

In order to determine a per capita residential flow and industrial and commercial flows, we have examined the 1980 water consumption records and the average daily flows at the water pollution control plant for 1978 - September 1981. Analyzing these two records we were able to estimate the sewage flows generated considering infiltration and inflow and water sales. Tables of flow records at the water pollution control plant are presented in Appendix 'B'. Based upon this analysis the following table has been prepared:

PRESENT FLOWS

LAND USE		WATER USE	INFILTRATION	TOTAL AVERAGE FLOWS
Residential	17,300 persons	186 l/c/d	91 l/c/d	277 l/c/d
Industrial	132 hectars	19,500 l/ha/d	7,850 l/ha/d	27,350 l/ha/d
Commercial	34 hectars	32,000 l/ha/d	7,850 l/ha/d	39,850 l/ha/d

These figures expressed as a total per capita average flow accounting for industrial and commercial usage as well as residential are found to be 570 lpcd. This is reasonable for urban development of the age and mix of the existing Georgetown urban area.

For future development we would expect that a substantial reduction in the infiltration and inflow can be made by good design and construction practises. An average per capita flow of 477 l/day (105 gal/day) should be expected for such new development. This would result in the following average flows for new development including infiltration.

PROJECTED FLOWS

LAND USE	TOTAL AVERAGE FLOW	
	Metric	Imperial
Residential	232 l/c/d	51 g/c/d
Industrial	23,025 l/ha/d	2050 g/c/d
Commercial	33,570 l/ha/d	2990 g/c/d

In the January 29, 1982 meeting of the Urban Area Review Steering Group the Ministry of the Environment indicated an average flow per capita of 570 lpd (125 gal/day) should be used in calculating new development flows. This has been used in our calculations of future needs, although we believe an examiniation of future flow records will indicate spare capacity of the water pollution control plant.

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Considering the Ministry's comments and assuming that the percentages of industrial and commercial development do not change dramatically from those existing, the projected average daily flows to the water pollution control plant would be:

ESTIMATED

AVERAGE FLOW TO THE

WATER POLLUTION CONTROL PLANT

	Population	m3/day	MIGD
Present Development	17,300 persons	9,830	2.163
Proposed Development to Present Planning Limitations	30,000 persons	17,045	3.75
Proposed Development to Proposed Planning Limit	40,000 persons	22,730	5.0

4.3 Water Pollution Control Plant Operations

4.3.1 General

Early in the study it became apparent that the key issue in this part of the evaluation would be the consideration of the plant's ability to discharge a nitrified effluent. The main thrust of the program therefore was to determine the maximum water pollution control plant loading rates which could be tolerated while still maintaining nitrification, that is biological oxidation of NH3-N in the activated sludge system. A summary of the operating data for plant B is found opposite.

The strength of the raw sewage entering the water pollution control plant is relatively weak as is shown in the data in Tables No. 1-1 to 1-4 in Appendix 'D'. The raw sewage characteristics were consistent and there appeared to be very little fluctuation due to industrial flows. The only expection to this was during condition 1 in January and February 1981 when a concentrated waste strength stream high in soluable BOD_5 was noted. We believe that this was due to the discharging of leachate from the Municipal landfill site into the sanitary sewer system.

(Opposite 4.4)

SUMMARY TABLE OF OPERATION

OF PLANT B

		CONDITION 1	CONDITION 2	CONDITION 3	CONDITION 4
PRIMARY EFFLUENT					
Q	m^3/d	4,000	9,000	5,600	10,000
SS	mg/L	88	97	79	112
BOD_5	mg/L	100	87	67	107
NH_3-N	mg/L	13	18.7	12.3	14.6
TKN	mg/L	25.6	29.5	21.9	24.3
AERATION TANK					
D.O.	mg/L	6.7	6.3	3.7	2.2
MLSS	mg/L	2,662	3,989	3,992	5,255
MLVSS	mg/L	1,437	2,274	2,076	2,730
TEMPERATURE	°C	10	12	18	18
SECONDARY EFFLUENT					
SS	mg/L	8	8	5	9
BOD_5-Tot	mg/L	22	13	8	15
BOD_5-Sol	mg/L	9	3	3	4
NH_3-N	mg/L	0.9	13.3	0.4	3.5
CHLORINE CONTACT CHAMBER EFFLUENT					
pH	mg/L	7.6	7.6	7.3	7.1
SS	mg/L	3	4	3	6
BOD_5-Tot	mg/L	4	4	3	4
BOD_5-Sol	mg/L	2	2	2	2
NH_3-N	mg/L	0.8	12.5	0.1	3.1
TKN	mg/L	3.6	15.1	2.5	6.0
P-Tot	mg/L	1.0	0.8	0.8	0.6
P-Sol	mg/L	0.8	0.7	0.8	0.5
ACTIVATED SLUDGE LOADING					
AT-load(kg BOD_5 /day)		400	783	375	1,070
(kg BOD_5 /m^3/day)		0.26	0.51	0.24	0.69
MLVSS	mg/L	1,437	2,274	2,076	2,730
F/M (kg BOD_5/kg MLVSS/day)		0.18	0.22	0.12	0.25
kg W.A.S. Solids/day		-	1,603	984	2,341
SRT (days)		-	3.8	6.2	3.5
HRT in aeration (hrs)		9.2	4.1	6.6	3.7
Nitrification		O.K	FAILED	O.K	MARGIONAL

Throughout the study, the primary treatment section's of Plant A and Plant B functioned well. Consistent quality primary effluent was obtained. The hydraulic loading on both primary treatment systems was found to be relatively low.

Throughout the entire study period excellent removal of the BOD_5 was provided by the plant. Even when all the sewage flow was directed to plant B, very good BOD_5 removal was noted.

Very consistent removal of suspended solids was also noted throughout the study period. The hydraulic loading on the secondary clarifiers were low and the effluent filter acted as a very effective polishing unit.

The results of the plant study indicate that effective precipitation and subsequent removal of phosphorus can be maintained in the plant. The present effluent requirement of 1 mg/L total phosphorus is routinely met. During March and April 1981, tests were conducted to determine whether a further reduction could be consistently met. The ferric chloride dosage was increased from the usual 10-12 mg/L to 15 and then 20 mg/L. The soluable phosphorus was reduced to 0.1 to 0.2 mg/L and the total phosphorus concentration was reduced to a level of less than 0.3 mg/L.

Nitrification of the effluent was obtained during certain operating conditions and not during others. We noted that almost complete nitrification was achieved during conditions 1 and 3 when all the units of the water pollution control plant were in service. When all the sewage flow was directed through Plant B during condition 2, i.e. during the winter months of January-February the nitrification process failed rapidly. During the warmer summer conditions higher flows were directed through plant B for condition 4 and the results show that nitrification began to deteriorate even though it did not fail completely. Based upon these experiences we believe to nitrify to the levels required it will be necessary to provide additional aeration tank capacity.

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The results of the plant study indicate that almost complete nitrification can be achieved during cold weather periods if the following inter-related conditions are met:

1. Aeration tank organic loading not greater than 0.25 kilograms of BOD_5 per cubic metre at design conditions.
2. Food to microrganism ratio of 0.15 kilograms BOD_5 per kilogram of mixed liquer volatile suspended solids per day.
3. Solids retention times of 6 to 7 days.

The existing aeration tank and the secondary clarifiers in plant A can treat a primary effluent flow of approximately 6,800 m^3/day (1.5 MIGD). The existing aeration tank capacity in plant B is restricted to treat a primary effluent of approximately 4,800 m^3/day (1.1 MIGD) even though the area of the second clarifier is very large and could handle flows of up to 45,500 m^3/day (10 MIGD).

Our investigation of the digestion facilities at the water pollution control plant confirmed the recommendations of the Simcoe Engineering Report on the Digester System. Essentially the existing anaerobic digester functions mainly due to an extremely low loading rate. However, the mixing equipment on the roof and gas equipment is completely outmoded and the digester tank leaks. Major renovation or replacement work will be required to update this unit process equipement.

Throughout the study it was continuously apparent that major improvements in the plant metering and sampling equipment is required.

4.4 Rogers Creek Monitoring Program

4.4.1 General

During the survey period various modes of operation of the Water Pollution Control Plant were being examined. The Water Pollution Control Plant was only operated under normal load conditions during the period October 9, 1980 to February 12, 1981 and June 24 to August 20, 1981. The following discussion is related to these periods only and is divided into three sections.

The first section addresses the existing water quality in the creek for present Water Pollution Control Plant loadings as measured. The next section predicts the creek water quality for the designated 7Q20 creek flows, based on extrapolating the measurements of the existing effluent and creek water quality, and the third section examines the effects on the creek water quality of a discharge from the plant of up to 0.27 m^3/s (5 MGD).

4.4.2 Rogers Creek Water Quality Existing Conditions

The plant effluent quality based on 24 hour composite samples indicates the existing effluent water quality and the provincial objectives as follows:

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	PLANT EFFLUENT QUALITY MEASURED	PLANT EFFLUENT FLOW m^3/sec	WATER QUALITY OBJECTIVE
Suspended Solids	mean 3.0 mg/l (1.0 to 6.0)	0.106	mean 9.9 mg/l
Total BOD	mean 4.1 mg/l (1.0 to 13.0)	0.106	function of the receiving water
Ammonia	Winter (Dec to Feb Apx.B Table 4-1) Mean 1.27 mg/l (0.22 to 3.92) pH mean 7.5 (7.3 to 7.8) Temp mean 8°C (6 to 12) (Apx B Table 2-1) Mean Un-ionized* 0.0061mg/l Max Un-ionized* 0.053mg/l	0.091	0.02 mg/l un-ionized
	Summer (June to Aug Apx B Table 4-3) Mean 0.1 mg/l (less 0.1 to 0.5) pH mean 7.3 (6.7 to 7.7) Temp mean 18.2°C (16 to 20) (Apx B Table 2-3) Mean Un-ionized* 0.0007mg/l Max Un-ionized* 0.0098mg/l	0.108	0.02 mg/l un-ionized
Total Phosphorus	ice free period mean 0.94 mg/l (0.25 to 2.1)	0.106	0.30 mg/l
Total Residual Chlorine	mean 0.215 mg/l (0.039 to 0.472)	0.106	0.002 mg/l
Fecal Coliforms	geometric mean 300 org/100 ml	0.106	1000 org/100 ml

* The un-ionized portion of the ammonia as a function of pH and temperature was computed as follows:
$R = 1.0/(1+10^X)$ where $X = (pka - pH)$ and
$pka = 0.09018+(2729.92/(Temp + 273.2))$
(MOE Water Resources Paper 14, 1980 p.9)

The present effluent exceeds the provincial water quality objective for total phosphorus and total residual chlorine and for ammonia in the winter period.

The creek survey data (Tables 1 to 6) downstream of the plant effluent discharge indicates the following water quality and the provincial objectives:

WATER QUALITY PARAMETER	CREEK	CREEK FLOW m^3/sec	WATER QUALITY OBJECTIVE
Suspended Solids	mean 18.1 mg/l	0.453	16.4 mg/l
Total Dissolved Solids	mean 900 mg/l	0.453	730 mg/l
Ammonia	Winter (Jan 21 & 29 Feb 5 & 12 Apx C Table 1) mean 0.88 mg/l (0.5 to 1.23) pH mean 7.95 (7.1 to 8.7) Temp 0.33°C (-0.5 to 1.0) Mean Un-ionized* 0.0066 mg/l Max Un-ionized* 0.053 mg/l	0.475	0.02 mg/l un-ionized
Total Phosphorus	ice free period mean 0.183 mg/l (0.075 to 0.362)	0.453	0.03 mg/l
Total Residual Chlorine	mean 0.028 mg/l (0.0 to 0.118)	0.453	0.002 mg/l
Dissolved Oxygen	Summer (June to Aug) mean 8.0 mg/l (7.5 to 8.3)	0.436	5 mg/l

The water quality degradations due to the existing Water Pollution Control Plant discharge at creek flows of approximately 0.45 m^3/s are suspended solids; total dissolved solids; total phosphorus; total residual chlorine and occasionally ammonia.

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4.4.3 Predicted Creek Water Quality for the Designated 7Q20 Flows

The measured creek water quality parameters were plotted as functions of creek flow in Figures 4 to 10, then extrapolated to the designated 7Q20 flows of 0.17 m^3/s in the winter and 0.144 m^3/s in the summer. The other water quality parameters not presented in the figures were extrapolated on the basis of the flow ratios during the surveys and 7Q20 flows. Using these extrapolations the creek water quality for the 7Q20 flows is predicted as follows:

PARAMETER	CREEK	CREEK FLOW m^3/sec	WATER QUALITY OBJECTIVE
Ammonia	Winter (Apx C Fig 4) Mean 1.0 mg/l Max 1.35 mg/l pH mean 7.95 (7.1 to 8.7) Temp mean 0.33 (-0.5 to 1.0) Mean Un-ionized 0.0075 mg/l Max Un-ionized 0.0582 mg/l	0.17	0.02 mg/l un-ionized
	Summer (Apx C Fig 4) Mean 0.25 mg/l pH mean 7.96 (7.9 to 8.35) Temp mean 18.4°C Mean Un-ionized 0.0078 mg/l	0.144	0.02 mg/l un-ionized
Total Phosphorus	mean 0.62 mg/l max 1.1 mg/l	0.144	0.03 mg/l
Total Residual Chlorine	mean 0.088 mg/l max 0.371 mg/l	0.144	0.002 mg/l
Dissolved Oxygen Deficit	mean 0.8 mg/l max 1.4 mg/l	0.144	3.8 mg/l

From these data it is observed that the water quality in Rogers Creek for the existing Water Pollution Control Plant discharge quality and 7Q20 flows that the Provincial objectives for ammonia, total phosphorus and total residual chlorine, are expected to be exceeded.

In the tables of the creek survey data upstream of the Water Pollution Control Plant discharge it is noted that the background stream concentrations exceed the water quality objectives for ammonia at creek flows of 0.475 m^3/s. These high background levels are incorporated into the method used to predict the creek water quality for 7Q20. Consequently the creek estimates are higher than expected for the effluent discharge. It is also observed that the creek ammonia concentrations decrease due to the existing Georgetown effluent discharge.

4.4.4 Predicted Creek Water Quality for an Effluent Discharge of Up to 0.26 m^3/s (5 MGD)

The treated effluent will achieve the water quality objectives of 0.02 mg/l un-ionized ammonia and be dechlorinated. The total phosphorus concentrations will not exceed the 0.3 mg/l required by the Ontario Ministry of the Environment (letter dated 30 December 1981). The other water quality parameter of interest is predicted to be as follows for 7Q20:

PARAMETER	EFFLUENT	CREEK FLOW m^3/sec	WATER QUALITY OBJECTIVE
Dissolved Oxygen Deficit	mean 2.0 mg/l max 3.5 mg/l	0.144	3.8 mg/l

The objectives are achievable with the modifications to the unit processes set out in Chapter 7.

CHAPTER 5

CONCLUSIONS

CHAPTER 5

CONCLUSIONS

5.1 Background

From our review of previous reports and documents and through discussions with the Region and Town operating and supervising personnel we believe that steps have been taken to implement the recommendations of the Proctor and Redfern and Simcoe Engineering reports. Additional remedial work is required particularly with respect to the Silver Creek Trunk Sewer which will be further expanded upon in the Conclusions relating to the sewer system survey in Appendix 'A'.

Independant of the recommendations of this study we understand that the Region has initiated a study to review the status of the digesters at the Georgetown Water Pollution Control Plant and have planned for the construction or renovation of the existing facility in 1983.

5.2 Sewer System Survey

5.2.1 Sewer Systems Study

1. Data from the monitoring of major storm events and high flow conditions has not shown specific areas of high infiltration.

2. The flow meter installed at the Water Pollution Control Plant provided data that indicates that whenever there is rain, whatever quantity, there is a high peak on that particular date. Therefore infiltration is evident in the collection system. The infiltration of the collection system is, however, widespread throughout the Georgetown area.

5-2

3. Dye tests done in arbitrary locations throughout Georgetown showed most house roof drain connections to be connected to the sanitary sewer system.

5.3 Water Pollution Control Plant Monitoring

The following are the conclusions from our review of the operation of the Georgetown Water Pollution Control Plant over the past year and our review of the previous years operations.

1. The available plant facilities are well operated at the present time and acceptable quality effluent is routinely produced at the plant.

2. Under all conditions observed during this study the removal of BOD_5 from the waste water was effectively achieved.

3. Under all conditions observed during the study removal of suspended solids was effectively achieved. The capture of suspended solids is limited only by the hydraulic capabilities of the tertiary filter.

4. Under all conditions observed during the study, removal of phosphorus was achieved. Results of the tests show a total phosphorus concentration of less than 0.3 mg/l was achieved as long as the water pollution control plant effluent was filtered and the chemical dosage was adequate.

5. The aeration tank capacity of the water pollution control plant limits the ability of the existing facility to nitrify the effluent under peak load conditions.

6. In general the Georgetown water pollution control plant has the potential to achieve effluent of a quality sufficient to meet the guidelines for the receiving stream required by the Ministry in their letter to Gore & Storrie Limited dated December 30, 1981 with modifications and additions to certain sections of the plant:

    a) Inlet works
    b) Primary clarifiers
    c) Aeration tanks
    d) Effluent filters
    e) Dechlorination facilities.

7. The existing plant capacity is limited by its ability to provide a proper nitirified effluent to meet the requirement of the Ministry of the Environment to prevent degradations in the Rogers Creek flows above a plant capacity of 11,364 m^3/day (2.5 MIGD).

5.4 Stream Monitoring

For a Water Pollution Control Plant loading of 0.26 m^3/s with proper nitrification, dechlorination as well as phosphorus removal to 0.3 mg/l, the Provincial water quality objectives will be achieved for all parameters except total phosphorus, suspended and total dissolved solids for a creek flow of 0.144 m^3/s (i.e. the summer 7Q20). The total phosphorus requirement has been set at a maximum of 0.3 mg/l and can be met with a higher chemical dosage.

CHAPTER 6

RECOMMENDATIONS

CHAPTER 6

RECOMMENDATIONS

6.1 General

Appendix 'D' contains the background reports for Stage II Halton Hills, Georgetown Urban Area Review prepared by Muncipal Planning Consultants. Population and development senarios are discussed in this report and are related to the capabilities of the sewage treatment system to accomodate these flows.

6.2 Sewer System & Pumping Stations

6.2.1 Sewer System

In the course of carrying out the monitoring program of the sewage system we found that certain areas require immediate attention by the Region.

These areas were listed in a letter to the Region dated 16 January 1981. The listing of the areas is included in Appendix 'A' of this report.

There are high instantaneous flows at the plant as a result of depressed manhole covers and poor manhole construction in the main trunk line coming to the plant through the river valley. To reduce infiltration in the system along this section requires raising of manhole covers and making repairs to existing manholes along the main trunk line. This sewer also requires cleaning to remove accumulations of sludge and gravel noted in the inspection.

6-2

6.3 Water Pollution Control Plant Monitoring Program

The terms of reference for the study set out the following objectives:

A. The study is to indicate the increased equivalent population, if any, which could be permitted with the existing pollution control plant and

B. To indicate further increases in the equivalent population which may be permitted using additional wastewater purification techniques permitted under current Ministry of the Environment regulations.

Our study has established the ability of the plant to consistently produce an acceptable effluent at the present plant loadings. Present and recommended full development population and flows are as follows:

A. 17,045 m^3/day
30,000 persons (the current Ministry of the Environment certificate)

B. 22,730 m^3/day
40,000 persons - considering the estimated future rate of flow as required by the Ministry of the Environment.

Chapter 7 sets out the necessary alterations and extensions to the treatment facilities to reach the above objectives.

We believe we have demonstrated that the Georgetown water pollution control plant can with the recommended alterations and modifications meet the requirements of the Ministry of the Environment.

CHAPTER 7

WATER POLLUTION CONTROL PLANT RECOMMENDED WORKS

CHAPTER 7

WATER POLLUTION CONTROL PLANT

RECOMMENDED WORKS

7.1 Introduction

The existing water pollution control plant is located in the Town of Georgetown and discharges into Rogers Creek approximately 1600 metres upstream of its confluence with the Credit River.

The Creek has a 7Q20 flow in the summer of 0.144 m^3/sec and in the winter of 0.170 m^3/sec. At low flow conditions the dilution ratio, that is stream flow to effluent flow, is less than 1:1 and this condition imposes more stringent effluent requirements than would be necessary for treatment facilities discharging into larger receiving bodies such as a lake or large river.

The Ministry of the Environment recognizing the lack of flow for effluent dilution and assimilation and having considered the upstream water quality have requested the following effluent requirements, the unionized ammonia concentration be less than or equal to the Provincial Water Quality Objectives, the total chlorine residual be less than 0.002 mg/l, the BOD_5 be less than 5 mg/l and the total phosphorus be less than 0.3 mg/l.

The plant study demonstrated that the above criteria can be met and we have prepared the following recommendations based on a treatment plant design flow of 22,730 m^3/day and a population of 40,000 persons.

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7.2 Existing Plant

The first treatment facility constructed at the site in 1959 is referred to as Plant A and is limited in the amount of flow it can treat by the size of its secondary clarifiers. It is proposed that this plant be limited to not more than 6,800 m^3/day and the second plant called plant B be expanded to accomodate the projected increased flows.

7.3 Raw Sewage Flow and Waste Strength Concentrations

Present and projected flows for both average and peak wet weather conditions used in developing the design parameters for the treatment facility are as follows:

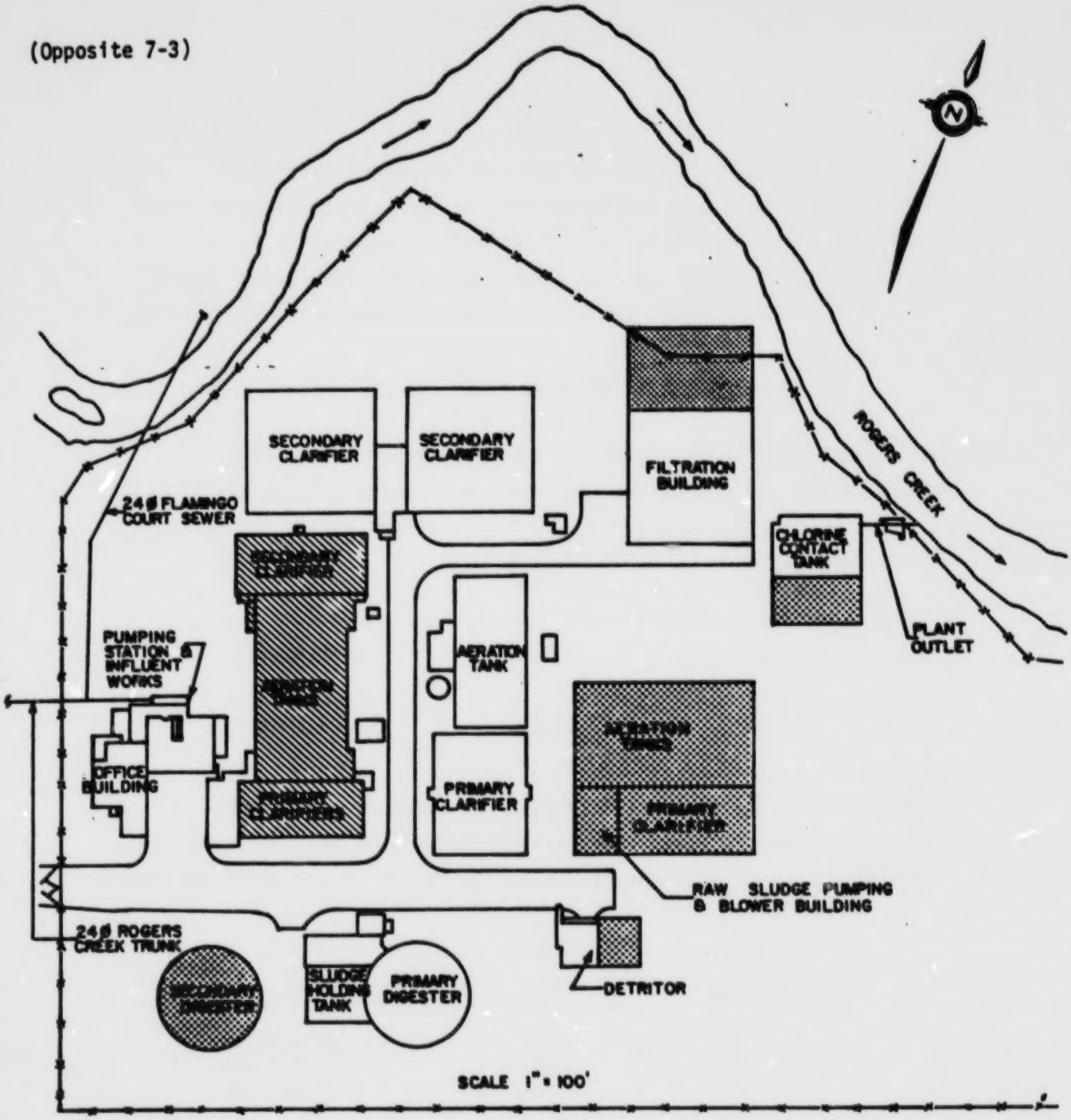
	Present m^3/day	Projected m^3/day
Average Flow	10,000	22,730
Maximum Wet Weather Flow	30,000	56,825

During the study of the treatment facility in 1980 and 1981 an extensive sampling program was carried out. Present and proposed design average BOD_5, SS, P in the raw sewage concentrations are as follows:

	Present	Proposed
	mg/l	mg/l
Average SS	187	200
Average BOD	138	150
Average P	5.47	6.0

Ammonia concentration however in the primary effluent averaged 15 mg/l and we have used this value in our design considerations of the biological treatment system.

(Opposite 7-3)



LEGEND

EXISTING WORKS

PLANT A

PLANT B

PROPOSED WORKS FOR TOTAL PLANT CAPACITY 20.9 m^3/d

HALTON HILLS (GEORGETOWN)
URBAN AREA REVIEW
GEORGETOWN SEWAGE TREATMENT PLANT

PROPOSED LAYOUT

GORE & STORRIE LIMITED

In considering the proposed design wastewater concentrations, enforcement of the Region's sewer use by-law is a fundamental requirement.

The proposed increase in strength above the average determined in the 1981 sampling program reflects the Region's program to reduce infiltration where possible and a conservative position with respect to projected residential commercial and industrial development mix.

7.4 Plant Site

The plant site is located on Rogers Creek and is serviced by an access road from Mountainview Road. The site has sufficient area for the proposed plant expansion. However it would be desirable to attain the strip of land on the south side of Rogers Creek directly north of the present northern boundary line. This would permit a more effective plant layout.

The proposed plant layout is shown opposite. Note that the proposed expansion of the filtration building is to the north rather than to the south and assumes that the land to the north will be made available. Alternate configurations may be examined in more detail in final design.

7.5 Inlet Works

Two major trunk sewers service the plant. The 24" Flamingo Court sewer from the north and a the 24" Rogers Creek trunk sewer from the west.

The capacity of the inlet sewer system in a nonsurcharge condition is about 80,000 m^3/day and the plant pumping station has this capacity also.

No major expansion of either the inlet sewers or the main plant pumping station is planned however, we would propose that the present grit works for plant B be improved, as during, high wet weather flows overflow at this particular chamber has occurred. In any case it will have to be

7-4

expanded to handle the peak flow rates now proposed. Also we would recommend raw sewage metering and sampling equipment be installed in the existing inlet works and the comminutor be replaced with a mechanical screen.

7.6 Primary Clarification

Due to hydraulic constraints in plant A and that plant's small secondary clarifiers we propose to base load plant A to 6,800 m^3/day.

Flows above 6,800 m^3/day will be treated in plant B and the expansion of that plant.

Expansion of the plant B primary clarifiers is necessary to provide treatment for peak wet weather flow up to 50,025 m^3/day.

As all primary effluent is ultimately given effluent filtration a clarifier overflow rate of 90 m^3/m^2/day has been used as a basis for sizing the primary clarifiers. It is proposed that one additional clarifier be provided with an area of 330 m^2. A single tank with a bridge type mechanism similar to the plant B clarifiers is proposed.

7.7 Biological Treatment

In order to reduce the BOD to 5 mg/l or less and to reduce the unionized ammonia concentration to the Provincial Water Quality objectives the aeration system must be expanded to allow removal of carbonaceous as well as nitrogenous organic matter. The latter material can be reduced in biological treatment by a process known as nitrification.

This study has shown that at solids retention times of 6 to 7 days, F/M values of 0.15 kg/BOD/kg MLVSS/day and organic loadings of .25 kg/m^3/day nitrification can be achieved.

The aeration section for plant A is satisfactory for nitrification providing flow rates do not exceed 6,800 m^3/day. Plant B aeration section will require expansion to handle flows above 4,800 m^3/day.

The designed BOD loading to the aeration tanks is approximately 1,820 kg/day and based on this loading an additional aeration tank volume of 3,600 m^3 is required. We propose that the aeration tank be located as shown in the plant layout and that rather than mechanical aeration, fine bubble aeration be used. Our reasoning for this change in process equipment is that the fine bubble diffused air system has less heat loss than mechanical aeration system and is more energy efficient in transferring oxygen.

7.8 Secondary Clarifiers

The existing secondary clarifiers have an area of 1,190 m^2 and at the proposed peak flow rate of 56,825 m^3/day the overflow rate would be 48 m^3/m^2/day.

With the effluent being filtered we believe that no additional secondary clarifiers are required.

7.9 Effluent Filtration

The existing effluent filtration system has a maximum capacity of 27,250 m^3/day and will require doubling of its present size. An additional 282 m^2 of effluent filter is required. The location of this facility is best to the north of the existing filter building due to the effluent filter inlet and outlet conduit arrangments. This however would encroach on the property to the north of the present plant site and it is desirable for the Region to acquire this property. Expansion to the south is possible however it would require a more difficult and costly connection of structures.

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7.10 Disinfection

In order to meet Provincial Water Quality Objectives with respect to bacteriological quality disinfection is required.

The most common and economic method of disinfection is that presently practiced at the plant, chlorination. Unfortunately chlorination raises the total chlorine residual in the effluent above Provincial Water Quality Objectives and dechlorination is required as per Ministry of the Environment letter of December 30, 1981.

To reduce the total chlorine residual we propose dechlorination utilizing sulphur dioxide.

The chlorine contact tank requires expansion to provide a minimum of 30 minutes detention for average flows and a doubling of the present facility is required. An additional 280 m^3/day of contact tank is necessary and could be provided south and adjacent to the existing structure. The facility would be equipped with a sulphur dioxide system to dechlorinate the effluent.

We would propose that the existing chlorine facility presently located in the administration building be abandoned and a new chlorination, dechlorination facility be provided as part of the effluent filtration building expansion.

7.11 Sludge Disposal

In the Simcoe Engineering study, disposal to agricultural land is proposed as the most effective sludge disposal method for the Georgetown plant. In order that a stabilized sludge is produced satisfactorily for land disposal it is essential that the existing digestion system be refurbished or replaced.

As this item is being studied as a separate engineering assignment we have not considered this unit process further with the exception of a general cost estimate.

7.12 Effluent Outlet Structure

We do not propose any changes to the existing effluent outfall structure diffusion across the river is not recommended as a corridor around the discharge structure is perferred for migrating fish.

7.13 Estimate of Cost

Based on the proposed works we have prepared general estimates. They are based on 1982 costs and should be adjusted as necessary when work proceeds.

Grit Tank Expansion, flow metering, sampling and mechanical screening	$ 500,000
Primary Clarifier	1,200,000
Aeration Tanks	1,800,000
Blower Building	500,000
Effluent Filtration	800,000
Chlorine Contact Tank	200,000
Chlorination-dechlorination Facility	100,000
Digestion Facility	$ 1,100,000**
Total Estimated Construction Cost	$ 6,200,000
Engineering and Contingencies (15%)	$ 930,000
TOTAL ESTIMATED CONSTRUCTION COST	$ 7,130,000

** The Region has requested that the digestion system be further studied however in an earlier letter to the Region from Gore & Storrie, dated the November 24, 1981, we estimated a 1981 cost for the digestion facility of $1,069,000 including engineering.

APPENDIX A

SEWERAGE SYSTEM SURVEY

TABLES

APPENDIX 'A'

SEWERAGE SYSTEM STUDY

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Pumping Station Investigations	A-28

GORE & STORRIE LIMITED
CONSULTING ENGINEERS
ESTABLISHED 1919
MEMBER C.E.O.-A.C.E.C.

1870 BAYVIEW AVENUE
TORONTO, ONTARIO M4G 3C2
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TELEX 06 23283

January 26, 1981

603.01

The Regional Municipality of Halton
Public Works Department
1151 Bronte Road
Oakville, Ontario
L6J 4Z3

Attention: Mr. H. H. Cripps, P.Eng.
Manager, Planning and Development

Gentlemen:

Re: Regional Municipality of Halton, Halton-Hills
(Georgetown) Urban Area Review

In the course of carrying out the above review of sewage system in Georgetown our survey crew has identified certain areas which require attention by the Region.

These areas are listed on the attached pages giving the area number, the manhole number, the location and problem. We would appreciate it if you would pass these comment sheets on to the appropriate operations and maintenance staff from the Region in order that they may make the necessary repairs and sewer flushings, to assist us in establishing the capabilities of the sewage system under the best of conditions.

In the meantime we will proceed with the study.

Yours very truly

N. D. Huggins, P.Eng.
Project Manager

NDH:mp

c.c. R. Mohamed
D.R. Morrier, P.Eng.
D. Barber

G & S PROJECT NO. 603.01

Date: January 16, 1981

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Regional Municipality of Halton
Halton Hills (Georgetown)
Urban Area Review

Date	Ref. Item	Location	Area #	MH #	Comments
/11/80	--	Mountainview & Delrex	2	232	MH - should be flushed as sludge has accumulated in drop pipe from west
	Ref.Item 12(r)	Delrex & Weber Drive	2	14	MH - overflow between the sanitary and storm sewer is plugged, thus confirming no cross connection exists between these sewers
	Ref.Item 7	Regan Cr. (in Gulley)	19	4	MH - evidence of surcharging to 600 mm below top of MH
	Ref.Item 7	Regan Cr. (in Gulley)	19	3	MH - evidence of surcharging to 600 mm below top
	Ref.Item 7	Regan Cr. (in Gulley)	19	2	MH - mortar at top is cracking, surcharging is evident to the top
	Ref.Item 7	Regan Cr. (in Gulley)	19	1	MH - no platform or benching makes monitoring difficult - surcharing above overflow pipe evident
	Ref.Item	WPCP	19	991	MH - lack of platform and benching makes flow monitoring difficult - surcharging evident to 900 mm below top
3/11/80		Mill St.	19	45	MH - 200 mm drop connection plugged
		Guelph East of Mill St.	19	40	MH - hole where pipe enters manhole requires plugging due to oversize opening around pipe
		Park East of Main St.	19	33	MH - evidence of surcharging
	Ref.Item 6	9th Line North of John St.	16	31	MH - flow from Mushroom Cannery - surcharging was witnessed with overflows through a 100 mm dia.overflow pipe located 300 mm below the top of the MH which discharges to the roadside.

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G & S PROJECT NO. 603.01

Date: January 16, 1981

Regional Municipality of Halton
Halton Hills (Georgetown)
Urban Area Review

Date	Ref. Item	Location	Area #	MH #	Comments
/11/80	Ref.Item 6	John St. P.S.	16	2	MH - over break around east pipe should be filled to prevent infiltration
		Moore Pk. P.S.	10	1	MH - very difficult to monitor in this MH due to lack of platform which should be placed above the effluent in the wet well
	Ref.Item 12(L)	Ewing St.	12	53	MH - concrete block in poor condition with numerous cracks especially at the mortar joints. - needs flushing
/11/80	Ref.Item 7	NW of WPCP	19	993	MH - frame and cover not mortared to top of MH - evidence of surcharging to 2.4 m above outlet.
		NW of WPCP	19	994	MH - cement block construction, voids around 750 mm outlet pipe, large stones in MH. - cover is not proper fit (too small)
		Park St.	19	34	MH - pipe from north requires flushing
	Ref.Item 12(M)	Princess Anne @ hospital	9	231	MH - 1 broken step iron, flushing required
	Ref.Item 12(M)	Princess Anne Drive	9	23	MH - Sludge deposits from pipe 600 mm above outlet, - flushing required
	Ref.Item 12(M)	Princess Anne Dr	9	20	MH - Sludge deposits from drop pipe entering from NE - periodical cleaning required
	Ref.Item 12(M)	Princess Anne Dr	9	19	MH - sewage flowing through outfall of drop connection indicating blockage in 200 mm drop connection - general build-up of sewage with sludge deposits - flushing required

Regional Municipality of Halton
Halton Hills (Georgetown)
Urban Area Review

Date	Ref. Item	Location	Area #	MH #	Comments
11/80		Main St.	7	2	MH - Brick construction, poor condition - concrete from benching is lodged in pipe downstream of MH. - sludge build up in sewers and MH, requires flushing
	Ref.Item 12(h)	Guelph St.	6	76	- flush line from 77 to 74 and 74 to 72 - flow is non-existent or very poor
	Ref.Item 12(g)	Maple Ave. & Guelph St.	6	72	MH - has been raised by 200 mm with concrete rings that should be mortared - flushing is required before any sewer monitoring can be performed - pipe from north is badly plugged with sludge
		Terry Court	5	4	MH - evidence of surcharging to 1.3 m above outlet
	Ref.Item 12(b)	Mountainview Rd.	4	274	MH - no benching, base has considerable sludge deposits and the pipe from the north is badly plugged with sludge
12/80		Main North of Maple	7	2	MH - sludge deposits at least 25 mm deep throughout manhole
		Park East of Main	8	33	MH - evidence of surcharging to with 2 metres of the top
	Ref.Item 12(b)	Ewing and Creek	12 & 13	53	MH - gravel and sludge deposits in outfall pipe - groundwater infiltration around NE pipe

FLOW MEASUREMENT TABLES

The following tables list the flows from the various areas identified in the 1977 Proctor and Redfern report.

To gain an appreciation from the areas studied measurements were taken at differing times and compared to the theoretical peak flow expected.

In most instances the flows measured were lower than the theoretical peak.

A-6

HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
1.	4.	27.	10.30AM	NOV19/80	5.0	74.	1.23	34.8	142.4
			7.20AM	DEC17/80	3.0	63.	0.50	14.1	142.4
			3.00PM	DEC18/80	6.5	90.	2.18	61.6	142.4
			11.30AM	JAN14/81	4.0	70.	0.84	23.9	142.4
			3.50PM	JAN14/81	4.0	102.	1.23	34.8	142.4
			2.20PM	FEB 4/81	3.0	52.	0.41	11.7	142.4
			9.50AM	FEB 5/81	5.0	72.	1.20	33.9	142.4
			12.35PM	FEB 5/81	4.0	86.	1.04	29.3	142.4
			8.50AM	FEB 6/81	4.0	60.	0.72	20.5	142.4
			1.05PM	FEB 6/81	7.0	85.	2.28	64.6	142.4
			6.25PM	FEB 6/81	3.0	63.	0.50	14.1	142.4
			10.30AM	FEB19/81	7.0	110.	2.95	83.6	142.4
			1.57PM	FEB19/81	3.5	65.	0.64	18.3	142.4
			9.47AM	APR14/81	5.5	94.	1.79	50.7	142.4
			12.50PM	APR14/81	3.5	65.	0.64	18.3	142.4
			11.16AM	APR23/81	3.0	57.	0.45	12.8	142.4

HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
2.	5.	15.	11.40AM	NOV12/80	3.0	83.	0.48	13.5	71.3
			2.30PM	NOV12/80	2.8	97.	0.50	14.3	71.3
			10.05AM	NOV19/80	3.0	102.	0.58	16.5	71.3
			7.05AM	DEC17/80	3.5	100.	0.71	20.2	71.3
			2.55PM	DEC18/80	2.8	100.	0.52	14.7	71.3
			11.40AM	JAN14/81	2.4	84.	0.35	9.9	71.3
			3.55PM	JAN14/81	2.0	54.	0.17	4.9	71.3
			10.50AM	FEB 4/81	2.8	88.	0.46	12.9	71.3
			1.25PM	FEB 4/81	2.4	78.	0.32	9.2	71.3
			9.40AM	FEB 5/81	3.0	86.	0.49	14.0	71.3
			8.55AM	FEB 6/81	2.8	73.	0.38	10.7	71.3
			12.40PM	FEB 6/81	2.8	75.	0.39	11.0	71.3
			6.35PM	FEB 6/81	3.0	100.	0.57	16.2	71.3
			10.22AM	FEB19/81	4.5	96.	0.97	27.6	71.3
			1.56PM	FEB19/81	3.0	95.	0.54	15.4	71.3
			9.45AM	APR14/81	4.0	106.	0.91	25.9	71.3
			12.45PM	APR14/81	3.0	106.	0.61	17.2	71.3
			11.24AM	APR23/81	4.0	91.	0.78	22.2	71.3

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HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
2.	232.	8.	11.40AM	NOV12/80	3.0	83.	0.33	9.2	69.4
			12.00	DEC17/80	1.5	86.	0.13	3.6	69.4
			2.50PM	DEC18/80	1.8	70.	0.13	3.8	69.4
			10.20AM	JAN14/81	1.7	104.	0.18	5.2	69.4
			4.00PM	JAN14/81	1.0	70.	0.06	1.6	69.4
			1.15PM	FEB 4/81	1.9	82.	0.17	4.8	69.4
			9.30AM	FEB 5/81	2.1	84.	0.20	5.7	69.4
			11.00PM	FEB 5/81	2.0	75.	0.17	4.8	69.4
			9.05AM	FEB 6/81	2.0	80.	0.18	5.1	69.4
			1.15PM	FEB 6/81	2.1	85.	0.20	5.8	69.4
			2.05PM	FEB 6/81	2.0	80.	0.18	5.1	69.4
			6.45PM	FEB 6/81	3.0	90.	0.35	10.0	69.4
			10.15AM	FEB19/81	3.0	100.	0.39	11.1	69.4
			1.45PM	FEB19/81	2.5	104.	0.32	9.0	69.4
			9.55AM	APR14/81	2.0	65.	0.15	4.1	69.4
			12.55PM	APR14/81	1.5	70.	0.10	2.9	69.4
			11.30AM	APR23/81	2.5	71.	0.22	6.1	69.4

HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	THEO. PEAK (l/s)
4.	1.	18.	7.30AM	NOV19/80	4.5	25.	0.28	8.0	60.9
			11.25AM	DEC17/80	6.0	34.	0.58	16.3	60.9
			1.10PM	DEC18/80	6.5	35.	0.66	18.7	60.9
			10.15AM	JAN14/81	7.0	32.	0.67	18.9	60.9
			2.50PM	JAN14/81	7.0	40.	0.83	23.6	60.9
			12.10PM	FEB 4/81	7.0	32.	0.67	18.9	60.9
			9.00AM	FEB 5/81	6.9	28.	0.57	16.2	60.9
			8.25AM	FEB 6/81	7.0	27.	0.56	15.9	60.9
			4.10PM	FEB 6/81	6.6	28.	0.54	15.3	60.9
			6.15PM	FEB 6/81	6.0	25.	0.42	12.0	60.9
			10.37AM	FEB19/81	8.0	39.	0.97	27.5	60.9
			2.06PM	FEB19/81	7.0	30.	0.63	17.7	60.9
			8.05AM	APR14/81	7.1	38.	0.81	22.9	60.9
			11.40AM	APR14/81	7.0	17.	0.35	10.0	60.9
			8.20AM	APR23/81	6.0	28.	0.47	13.4	60.9

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HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
5.	4.	12.	8.40AM	NOV19/80	1.0	155.	0.16	4.5	53.5
			7.40AM	DEC17/80	1.5	150.	0.28	7.9	53.5
			2.35PM	DEC18/80	1.0	142.	0.15	4.1	53.5
			10.50AM	JAN14/81	1.0	130.	0.13	3.8	53.5
			3.10PM	JAN14/81	0.8	72.	0.05	1.5	53.5
			9.50AM	FEB 4/81	0.9	145.	0.13	3.6	53.5
			12.35PM	FEB 4/81	0.9	115.	0.10	2.9	53.5
			7.10AM	FEB 5/81	1.2	160.	0.21	6.1	53.5
			11.40AM	FEB 5/81	1.2	118.	0.16	4.5	53.5
			8.35AM	FEB 6/81	1.3	150.	0.23	6.4	53.5
			1.50PM	FEB 6/81	0.8	110.	0.08	2.3	53.5
			4.40PM	FEB 6/81	1.2	130.	0.17	4.9	53.5
			10.50AM	FEB19/81	1.0	120.	0.12	3.5	53.5
			2.15PM	FEB19/81	1.7	140.	0.31	8.8	53.5
			9.22AM	APR14/81	2.5	135.	0.52	14.9	53.5
			12.27PM	APR14/81	1.5	50.	0.09	2.6	53.5
			11.04AM	APR23/81	2.0	60.	0.17	4.8	53.5

HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
6.	3.	12.	8.55AM	DEC17/80	2.3	80.	0.28	7.8	47.3
			2.25PM	DEC18/80	1.5	85.	0.16	4.5	47.3
			11.00AM	JAN14/81	1.5	56.	0.10	2.9	47.3
			3.15PM	JAN14/81	1.5	65.	0.12	3.4	47.3
			10.00AM	FEB 4/81	1.2	71.	0.10	2.7	47.3
			12.45PM	FEB 4/81	1.4	75.	0.13	3.6	47.3
			7.25AM	FEB 5/81	1.3	66.	0.10	2.8	47.3
			11.50AM	FEB 5/81	1.8	70.	0.17	4.8	47.3
			12.00	FEB 6/81	1.2	72.	0.10	2.7	47.3
			11.03AM	FEB19/81	2.5	86.	0.33	9.5	47.3
			12.22PM	FEB19/81	4.5	60.	0.53	15.0	47.3
			8.15AM	APR14/81	6.1	36.	0.47	13.4	47.3
			11.50AM	APR14/81	5.9	15.	0.19	5.4	47.3
			9.00AM	APR23/81	2.5	91.	0.35	10.0	47.3

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HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
7.	2.	10.	8.30AM	NOV19/80	3.0	56.	0.25	7.2	22.9
			9.05AM	DEC17/80	2.0	30.	0.08	2.2	22.9
			2.20PM	DEC18/80	0.8	32.	0.02	0.6	22.9
			11.05AM	JAN14/81	1.5	32.	0.05	1.5	22.9
			3.25PM	JAN14/81	1.0	35.	0.03	0.9	22.9
			12.50PM	FEB 4/81	0.4	90.	0.02	0.6	22.9
			7.35AM	FEB 5/81	0.5	84.	0.03	0.8	22.9
			11.55AM	FEB 5/81	0.6	100.	0.04	1.2	22.9
			10.05PM	FEB 6/81	0.5	95.	0.03	1.0	22.9
			4.50PM	FEB 6/81	0.4	80.	0.02	0.5	22.9
			5.15PM	FEB 6/81	3.0	35.	0.16	4.5	22.9
			11.15AM	FEB19/81	0.7	105.	0.06	1.6	22.9
			8.20AM	APR14/81	0.8	105.	0.07	2.0	22.9
			11.55AM	APR14/81	1.0	45.	0.04	1.2	22.9
			8.56AM	APR23/81	1.0	60.	0.06	1.6	22.9

HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
8.	33.	8.	3.00PM	NOV18/80	0.8	26.	0.02	0.4	9.1
			9.15AM	DEC17/80	1.3	54.	0.07	1.8	9.1
			2.15PM	DEC18/80	1.0	32.	0.03	0.7	9.1
			11.15AM	JAN14/81	0.8	30.	0.02	0.5	9.1
			3.30PM	JAN14/81	0.5	15.	0.00	0.1	9.1
			10.15AM	FEB 4/81	0.8	15.	0.01	0.3	9.1
			1.00PM	FEB 4/81	0.6	15.	0.01	0.2	9.1
			7.45AM	FEB 5/81	0.7	20.	0.01	0.3	9.1
			12.00	FEB 5/81	0.8	28.	0.02	0.5	9.1
			12.15PM	FEB 6/81	0.8	26.	0.02	0.4	9.1
			5.05PM	FEB 6/81	1.0	20.	0.02	0.5	9.1
			11.20AM	FEB19/81	1.3	50.	0.06	1.6	9.1
			2.30PM	FEB19/81	1.0	35.	0.03	0.8	9.1
			8.25AM	APR14/81	1.0	45.	0.04	1.1	9.1
			11.56AM	APR14/81	0.5	20.	0.01	0.2	9.1
			8.46AM	APR23/81	1.0	23.	0.02	0.5	9.1

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HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
9.	2.	10.	10.00AM	DEC17/80	2.5	62.	0.22	6.1	39.9
			2.10PM	DEC18/80	2.5	60.	0.21	5.9	39.9
			11.20AM	JAN14/81	2.2	54.	0.16	4.5	39.9
			3.35PM	JAN14/81	2.0	52.	0.13	3.7	39.9
			8.30AM	FEB 4/81	2.4	62.	0.20	5.8	39.9
			1.05PM	FEB 4/81	3.0	54.	0.24	6.9	39.9
			7.55AM	FEB 5/81	3.0	54.	0.24	6.9	39.9
			12.10PM	FEB 5/81	3.0	56.	0.25	7.2	39.9
			12.20PM	FEB 6/81	2.8	50.	0.21	5.8	39.9
			11.28AM	FEB19/81	2.8	75.	0.30	8.5	39.9
			2.35PM	FEB19/81	2.3	62.	0.19	5.3	39.9
			8.28AM	APR14/81	3.0	62.	0.28	7.9	39.9
			11.58AM	APR14/81	3.0	62.	0.28	7.9	39.9
			8.45AM	APR23/81	3.0	55.	0.25	7.0	39.9

HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
10.	1.	12.	11.40AM	NOV18/80	2.4	68.	0.25	7.1	31.4
			11.45AM	DEC17/80	1.0	65.	0.07	1.9	31.4
			1.45PM	DEC18/80	1.5	80.	0.15	4.2	31.4
			9.15AM	FEB 4/81	1.0	75.	0.08	2.2	31.4
			3.00PM	FEB 4/81	1.6	35.	0.07	2.0	31.4
			8.30AM	FEB 5/81	0.8	110.	0.08	2.3	31.4
			8.05AM	FEB 6/81	1.5	115.	0.21	6.1	31.4
			10.40AM	FEB 6/81	1.6	100.	0.20	5.8	31.4
			2.55PM	FEB 6/81	1.1	60.	0.07	2.0	31.4
			5.50PM	FEB 6/81	1.6	70.	0.14	4.0	31.4
			11.54AM	FEB19/81	2.5	75.	0.29	8.3	31.4
			8.52AM	APR14/81	2.0	78.	0.22	6.2	31.4
			12.10PM	APR14/81	1.0	50.	0.05	1.5	31.4
			9.23AM	APR23/81	1.5	60.	0.11	3.2	31.4

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HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
11.	45.	12.	10.25AM	DEC17/80	1.0	98.	0.10	2.8	22.9
			1.55PM	DEC18/80	1.0	84.	0.09	2.4	22.9
			9.30AM	JAN14/81	1.5	80.	0.15	4.2	22.9
			2.35PM	JAN14/81	0.5	53.	0.02	0.6	22.9
			8.55AM	FEB 4/81	1.5	106.	0.20	5.6	22.9
			3.25PM	FEB 4/81	1.0	67.	0.07	1.9	22.9
			8.10AM	FEB 5/81	1.8	92.	0.22	6.3	22.9
			11.25AM	FEB 5/81	1.6	82.	0.17	4.7	22.9
			7.50AM	FEB 6/81	1.5	80.	0.15	4.2	22.9
			11.40AM	FEB 6/81	1.4	75.	0.13	3.6	22.9
			5.30PM	FEB 6/81	1.5	80.	0.15	4.2	22.9
			11.42AM	FEB19/81	1.0	105.	0.11	3.0	22.9
			2.48PM	FEB19/81	1.0	92.	0.09	2.7	22.9
			8.40AM	APR14/81	2.0	45.	0.13	3.6	22.9
			12.04PM	APR14/81	1.5	50.	0.09	2.6	22.9
			9.09AM	APR23/81	2.0	35.	0.10	2.8	22.9

HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
12.	53.	12.	11.20AM	NOV18/80	4.0	11.	0.08	2.3	9.6
			11.00AM	DEC17/80	1.5	55.	0.10	2.9	9.6
			1.35PM	DEC18/80	1.5	30.	0.06	1.6	9.6
			9.55AM	JAN14/81	1.3	60.	0.09	2.6	9.6
			2.05PM	JAN14/81	1.3	12.	0.02	0.5	9.6
			9.25AM	FEB 4/81	1.4	13.	0.02	0.6	9.6
			3.10PM	FEB 4/81	1.6	31.	0.06	1.8	9.6
			8.40AM	FEB 5/81	1.3	47.	0.07	2.0	9.6
			12.15PM	FEB 5/81	1.3	45.	0.07	1.9	9.6
			8.10AM	FEB 6/81	1.5	60.	0.11	3.2	9.6
			5.45PM	FEB 6/81	1.5	45.	0.08	2.4	9.6
			12.05PM	FEB19/81	2.5	80.	0.31	8.8	9.6
			3.00PM	FEB19/81	2.3	70.	0.24	6.8	9.6
			9.04AM	APR14/81	1.5	70.	0.13	3.7	9.6
			12.16PM	APR14/81	1.5	50.	0.09	2.6	9.6
			9.38AM	APR23/81	1.5	55.	0.10	2.9	9.6

HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
14.	2.	10.	1.25PM	NOV18/80	1.5	130.	0.22	6.2	42.5
			10.45AM	DEC17/80	2.0	170.	0.43	12.3	42.5
			1.50PM	DEC18/80	1.5	120.	0.20	5.7	42.5
			9.40AM	JAN14/81	2.0	165.	0.42	11.9	42.5
			2.25PM	JAN14/81	3.9	140.	0.90	25.6	42.5
			9.05AM	FEB 4/81	1.5	150.	0.25	7.1	42.5
			3.20PM	FEB 4/81	1.7	150.	0.30	8.6	42.5
			8.15AM	FEB 5/81	1.4	96.	0.15	4.1	42.5
			11.30AM	FEB 5/81	1.5	126.	0.21	6.0	42.5
			7.55AM	FEB 6/81	1.2	90.	0.11	3.1	42.5
			11.50AM	FEB 6/81	1.5	140.	0.24	6.7	42.5
			5.35PM	FEB 6/81	1.5	105.	0.18	5.0	42.5
			11.46AM	FEB19/81	1.8	190.	0.42	11.8	42.5
			2.53PM	FEB19/81	1.5	134.	0.23	6.4	42.5
			8.46AM	APR14/81	2.0	175.	0.45	12.6	42.5
			12.07PM	APR14/81	2.0	125.	0.32	9.0	42.5
			9.14AM	APR23/81	3.0	130.	0.59	16.6	42.5

HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	THEO. PEAK (l/s)
15.	2.	10.	1.25PM	NOV18/80	1.0	98.	0.09	2.6	42.5
			10.45AM	DEC17/80	0.8	110.	0.07	2.1	42.5
			1.50PM	DEC18/80	1.0	55.	0.05	1.4	42.5
			9.40AM	JAN14/81	1.0	110.	0.10	2.9	42.5
			2.25PM	JAN14/81	0.8	104.	0.07	2.0	42.5
			9.05AM	FEB 4/81	0.8	120.	0.08	2.3	42.5
			3.20PM	FEB 4/81	0.8	82.	0.05	1.6	42.5
			8.15AM	FEB 5/81	1.5	125.	0.21	6.0	42.5
			11.30AM	FEB 5/81	1.5	122.	0.21	5.8	42.5
			7.55AM	FEB 6/81	1.2	90.	0.11	3.1	42.5
			11.50AM	FEB 6/81	1.0	105.	0.10	2.8	42.5
			5.35PM	FEB 6/81	1.1	100.	0.11	3.0	42.5
			11.46AM	FEB19/81	2.0	120.	0.31	8.7	42.5
			2.53PM	FEB19/81	2.0	138.	0.35	10.0	42.5
			8.46AM	APR14/81	2.0	85.	0.22	6.1	42.5
			12.07PM	APR14/81	1.5	120.	0.20	5.7	42.5
			9.13AM	APR23/81	1.5	62.	0.10	3.0	42.5

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HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
16.	2.	12.	11.00AM	NOV18/80	1.0	65.	0.07	1.9	42.5
			11.10AM	DEC17/80	2.5	158.	0.61	17.4	42.5
			1.20PM	DEC18/80	1.0	38.	0.04	1.1	42.5
			10.00AM	JAN14/81	2.0	70.	0.20	5.6	42.5
			2.00PM	JAN14/81	2.5	70.	0.27	7.7	42.5
			9.35AM	FEB 4/81	1.4	37.	0.06	1.8	42.5
			3.15PM	FEB 4/81	1.6	60.	0.12	3.5	42.5
			8.50AM	FEB 5/81	1.0	43.	0.04	1.2	42.5
			12.20PM	FEB 5/81	1.0	27.	0.03	0.8	42.5
			8.15AM	FEB 6/81	1.0	28.	0.03	0.8	42.5
			12.55PM	FEB 6/81	0.8	32.	0.02	0.7	42.5
			6.05PM	FEB 6/81	1.5	58.	0.11	3.1	42.5
			12.10PM	FEB19/81	1.8	70.	0.16	4.6	42.5
			3.09PM	FEB19/81	1.0	66.	0.07	1.9	42.5
			9.14AM	APR14/81	2.0	74.	0.21	5.9	42.5
			12.21PM	APR14/81	2.0	48.	0.14	3.8	42.5
			10.52AM	APR23/81	3.0	91.	0.46	13.0	42.5

HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
18.	40.	8.	2.00PM	NOV18/80	1.0	61.	0.05	1.4	5.7
			10.15AM	DEC17/80	1.0	56.	0.05	1.3	5.7
			2.00PM	DEC18/80	1.0	25.	0.02	0.6	5.7
			9.15AM	JAN14/81	1.0	36.	0.03	0.8	5.7
			2.40PM	JAN14/81	0.8	30.	0.02	0.5	5.7
			8.40AM	FEB 4/81	1.0	20.	0.02	0.5	5.7
			3.40PM	FEB 4/81	1.8	50.	0.10	2.7	5.7
			8.00AM	FEB 5/81	0.8	31.	0.02	0.5	5.7
			11.15AM	FEB 5/81	1.0	43.	0.04	1.0	5.7
			7.40AM	FEB 6/81	0.8	30.	0.02	0.5	5.7
			10.50AM	FEB 6/81	1.2	37.	0.04	1.1	5.7
			1.25PM	FEB 6/81	1.0	40.	0.03	0.9	5.7
			5.20PM	FEB 6/81	1.2	42.	0.05	1.3	5.7
			11.35AM	FEB19/81	1.8	70.	0.13	3.7	5.7
			2.40PM	FEB19/81	0.8	50.	0.03	0.8	5.7
			8.32AM	APR14/81	1.3	60.	0.07	1.9	5.7
			12.00	APR14/81	0.5	0.	0.00	0.0	5.7
			8.40AM	APR23/81	1.2	11.	0.01	0.3	5.7

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HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
19.	991.	24.	9.40AM	NOV12/80	16.7	72.	5.51	156.0	240.1
			8.20AM	DEC17/80	12.1	30.	1.56	44.2	240.1
			3.15PM	DEC18/80	9.0	20.	0.71	20.0	240.1
			12.00	JAN14/81	11.9	72.	3.67	103.9	240.1
			4.10PM	JAN14/81	8.0	32.	0.96	27.2	240.1
			10.35AM	FEB 4/81	9.0	36.	1.27	36.0	240.1
			1.45PM	FEB 4/81	12.1	57.	2.97	84.0	240.1
			9.15AM	FEB 5/81	8.7	30.	1.01	28.6	240.1
			7.10AM	FEB 6/81	9.5	33.	1.25	35.5	240.1
			12.25PM	FEB19/81	13.5	60.	3.58	101.4	240.1
			3.26PM	FEB19/81	13.5	60.	3.58	101.4	240.1
			10.05AM	APR14/81	11.5	46.	2.24	63.5	240.1
			1.05PM	APR14/81	12.5	68.	3.69	104.4	240.1
			12.00AM	APR23/81	11.0	31.	1.43	40.4	240.1

HALTON HILLS URBAN AREA REVIEW-(GEORGETOWN)

AREA	M.H.	DIA (ins)	TIME	DATE	DEPTH (ins)	VELOCITY (cm/s)	FLOW ACTUAL (cfs)	FLOW ACTUAL (l/s)	FLOW THEO. PEAK (l/s)
19.	1.	24.	9.55AM	NOV12/80	11.0	50.	2.30	65.2	260.5
			8.05AM	DEC17/80	6.0	96.	1.93	54.7	260.5
			3.15PM	DEC18/80	13.8	30.	1.84	52.1	260.5
			11.55AM	JAN14/81	10.0	45.	1.83	51.8	260.5
			4.05PM	JAN14/81	6.0	92.	1.85	52.5	260.5
			10.30AM	FEB 4/81	7.4	54.	1.46	41.3	260.5
			1.40PM	FEB 4/81	8.3	52.	1.64	46.5	260.5
			9.10AM	FEB 5/81	6.0	82.	1.65	46.8	260.5
			7.00AM	FEB 6/81	11.0	35.	1.61	45.6	260.5
			12.20PM	FEB19/81	14.0	50.	3.12	88.3	260.5
			3.20PM	FEB19/81	16.0	100.	7.30	206.6	260.5
			10.02AM	APR14/81	14.0	80.	4.99	141.3	260.5
			1.00PM	APR14/81	15.5	37.	2.60	73.7	260.5
			11.55AM	APR23/81	15.0	46.	3.12	88.2	260.5

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DETAILED SURVEY OF SEWERAGE SYSTEM

The following points are addressed in the terms of reference (ref. Appendix 'E') and were investigated in the field. The terms of reference number is noted in brackets.

(5) Dry weather sewage flows in the Roger's Creek and Flamingo Court trunk sewers are normal for the given area and equivalent population densities.

(5) Visual checks of the by-pass sewer at the plant were included in the monitoring program during periods of high flows.

(6) The Armstrong Avenue, John Street and the Moore Park Pumping Stations were included in the monitoring program. High flows were not experienced at these points during storm events and high flow conditions.

(7) Manhole #1, at the old S.T.P. site on the Rogers Creek trunk sewer could not be located by the Region staff. Manholes in this area were checked for possible overflow conditions. It was determined that overflows have not occured at these locations.

(12(a)) No apparent overflow condition exists between manhole #9 and the storm sewer in Area 1 on the Flamingo Court trunk.

(12(b)) Manhole #274 in area 18 is sealed tight eliminiating the possibility of surface drainage water entering at this point.

(12(c)) The sewer pipe at manhole #791, area 4 was at least half full with flow but downstream of this point very little if any flows were evident. It would appear, therefore, that there is a blockage in the pipe downstream of this manhole.

(12(d)) The catchbasin on Chipper Court has been disconnected from the sanitary sewer serving Area 5.

(12(e)) The catchbasin on Pauline Street has been disconnected from manhole #101 in Area 6.

(12(f)) Infiltration of the system between manhole 76 and manhole 77 in Area 6 is insignificant for periods of dry weather flow. The flat grade of the pipe in this area makes frequent flushing of the line necessary.

(12(g)) The top of manhole 72 has been raised approximately 0.3 metres but due to existing grades in the area the manhole is located in a low lying area subjecting it to possible flooding and infiltration.

(12(h)) There is no apparent infiltration of the cluster of manholes at the intersection of Hwy. 7 and Maple Avenue.

(12(i)) The Domtar sewage lagoon in Area 16 is no longer connected to the sanitary sewer system.

(12(j)) The manhole north of the CNR on the Rogers Creek trunk has a high volume of water in it. There is an apparent blockage downstream of this manhole with considerable gravel deposits in the pipe.

(12(k)) The brick work is flaking on manhole #2 in area 19.

(12(l)) The manholes in the Rogers Creek trunk sanitary sewer between Ewing Street and the CNR embankment are built up with cement blocks. Mortar joints between the blocks are cracked, some manhole frames and covers are not properly grouted into place and there are gravel deposits in sections of this sewer. All these factors could result in excessive infiltration into the system at this point.

(12(m)) The footing drain sump pump at the hospital has been disconnected, the connecting sewer on Princess Anne Drive is in good condition with the exception of sludge accumulations in the manhole drop pipes.

A-26

(12(n)) There was no problem evident at manhole 87 in area 9. The sewers in this area have been re-aligned recently but plans showing the new sewer routes were not available.

(12(o)) Manhole 22 on Princess Anne Drive has no apparent infiltration problem.

(12(p)) The Mill Street service area (area 9) and the Park Street service (area 8) show increased flows during wet weather indicating infiltration of the system in these areas. John Street service area (area 16) experienced normal dry and wet weather flows.

(12(q)) Manhole 274 in area 4 has had the top raised 0.3 metres with concrete rings.

The manhole frame and cover have not been mortared, there is not benching in the manhole, and there is considerable sludge deposits in the manhole partially blocking the pipe.

(12(r)) There is no evidence of an overflow between the sanitary and storm sewers at the intersection of Delrex Blvd. and Weber Driver.

(12(s) Manhole 17 in area 19 could not be located to determine flooding and infiltration conditions at this point. Manholes 51,52, and 53 have been discussed herein before.

(12(t)) The flows from the Meadow Glen Mushroom Company appeared to be about 3 times that identified by the Region in their previous survey. Flow measurements taken at different periods throughout the day indicate an average flow of 18.7 litres per second. The sanitary sewerage system can handle the average daily flows from the Mushroom Company but the method of intermittent discharges to the system results in periods of high flow during the day causing overflowing of the manhole. The mushroom barns do not discharge to the sanitary sewers. The wastes from the barns are treated in on site lagoons with no visible discharge.

PUMPING STATION INVESTIGATIONS

Armstrong Avenue Pumping Station

Investigation

The Armstrong Avenue pumping station was constructed about 1960 by Heslop Construction who was the developer for that area of the town. The station is of cast-in-place concrete of wet well/dry well type. The brick superstructure covers about two thirds the area of the dry well, sufficient to house the stair well, and the electrical equipment. The superstructure can be extended to cover the remainder of the dry well area which would provide area for a third pump to be installed.

The wet well is divided with entry to each section through a manhole at grade level. An access laddder is installed in each well. The sewer inlet to the station is about 3 feet below grade level. A barminutor was orginally installed in the inlet channel, however, this is unit was being removed and a standard bar rack being installed for removal of course materials from the sewage.

The pumps are Smart Turner with flexible shafting to the motors mounted on the grade level floor. The motors are Robbins and Myers, mounted on fabricated ring bases. The name plates on the motors and pumps have been removed. Some information on these pumps was obtained from Mr. Bob Rolfe of the Region; no further information was available.

The No. 1 pump is a 4" x 3" WIV, 1140 rpm, 400 Ipgm at 43 feet head. The No. 2 pump is a 5" x 4" WBV 69, head and capacity are not available. By calculation this pump may have a capacity of about 800 Ipgm at 43 feet head.

Eight inch suction pipes are provided through the wall with 6-inch gate valves at the wall. A blanked off 8-inch suction pipe is provided for a third pump installation. The 5" x 4" pump is located in the centre position.

A-28

The discharge piping from each pump is equipped with a 6-inch check valve and 6-inch gate valve. The check valves do not have counter weighted arms and recently the station experienced a pump failure when a check valve failed to close allowing the force main to drain back causing the pump to rotate in the reverse direction; when the wet well level rose to the start level the pump started and the shaft broke.

The pumps discharge into a 12-inch header pipe which is equipped with a 12-inch check valve. The velocity in the 12-inch pipe is low and the flap may not have sufficient opening to clear debris from the passageway. We question if this check valve is serving any purpose. There is a 12 x 12 x 8 wye in the header for the installation of the third pump. Both pumps in this station have clockwise rotation. The No. 1 pump equipped with a 10 horsepower 550 volt 3 phase 60 cycle motor and the No. 2 motor is 20 horsepower 550 volt 3 phase 60 cycle. Both pumps are equipped with packing glands. Although a seal water system was installed it is not in operation and the pumps are presently operating without seal water.

Float switches to control pump operation are installed over the wet wells. Originally they were outside, however, they found it necessary to construct a wooden box with hinged cover over the switches in order to maintain operation during winter weather.

The wet wells are equipped with a ventilation fan. We question if the fan provides proper ventilation as it appears to be over one section of the well only and the divider wall completely separates the two wells. The dry well is equipped with a small wall type ventilator located on the outside wall about 3 feet above grade. The 6" x 10" duct from the basement would not provide a proper amount of ventilation.

This station does not have stand-by power facilities and as a result an overflow pipe is provided to the nearby water course. We question if this will be allowed to continue by the Ministry.

We were advised that sludge truckers are dumping sludge into the manhole immediately upstream from the pumping station resulting in heavy sludges coming into the sewer system.

The station is located in the Armstrong Avenue industrial area which appears to be growing reasonably rapidly. The pump operating range in the wet well is approximately 3 feet with pump operation approximately five minutes out of every 15 minutes. Only the large pump was operating on the day of the visit.

The discharge pipe leaving the station is at the same level as the pumps.

Recommendations for Armstrong Ave. Pumping Station

The two existing check valves on the pump discharges should be removed and counter weighted check valves installed in their place. This would ensure proper valve closing and prevent drain back and reverse rotation of the pumps resulting in damage as was earlier experienced. The 12-inch check valve in the main header pipe should be removed and replaced with flanged pipe. It was noted that some flanges in the discharge piping are uni-flange type which are attached to the pipe with set-screws. This would facilitate the removal and reinstallation of check valves as the pipes could be readily shortened to suit the new valves.

The superstructure should be enlarged so that a diesel generator set can be installed to provide stand-by power. This would also enclose the area for the motor for the third pump when it is installed.

A review of the flows to the station will ascertain the pumping requirement. The projected flows may indicate a third pump or replacement of the No. 1 unit may be required to meet the flow conditions. As the system exists, the smaller pump may not provide the stand-by capacity requirements of the Ministry of the Environment.

A-30

The ventilation system should be brought up to standard. The dry well should have 6 air changes per hour. Wet well ventilation is not necessary as there is no man entry unless the well is taken out of service and then portable ventilation can be used.

A proper seal water system should be installed and placed into operation as soon as possible as the absence of clean flushing water to the packing glands will allow grit to become embedded in the packing resulting in shaft sleeve wear. We recommend the seal water be taken from the potable water system using a backflow preventer and pressure reducing valve.

The bar rack being installed by the Region should be about 2-inches clear opening so that only course material will be retained on the screen, the small pump is 4 x 3 and probably has a maximum solids handling capability of 2 to 2-1/2 inches.

A bubbler system for pump control should be considered if the float switches presently in use give trouble.

John Street Pumping Station

Investigation

This pumping station is a cast-in-place concrete structure of the wet well/dry well type with brick superstructure. The No. 1 pump is 6" x 4" Smart Turner with packing glands. The suction to the pump is 6-inch diameter with a Walworth plug valve for shut-off. The discharge increases from the pump nozzle to 6-inch diameter. A Jenkins check valve with weighted arms and a Walworth plug valve are in the line before connecting to the header pipe. Dresser couplings are in both suction and discharge piping of this unit.

The motor is Robbins and Myers 1760 rpm 550 volts 3 phas 60 cycle. This is a built-together unit with a common motor and pump shaft.

The discharge from the pump is horizontal into the discharge header of which the centre line is approximately 4-1/2 feet above the floor. There is no means of ready access around the equipment without crawling under piping.

The No. 2 pump is a recent replacement, a Smart Turner 6WJ-S model with centre-line discharge. The pump is rated at 900 US gpm at 120 feet head at 1750 rpm with 50 horsepower Brooks Compton Parkinson motor. This pump is equipped with mechanical seals with seal water being taken from the discharge piping, passed through a filter and returned into the seal. This method of providing seal water should be avoided if another water supply is available.

The centre-line of the new pump casing is higher than that of the original pump and the discharge pipe drops approximately 6-inches through a fabricated offset fitting to the centre-line of the existing header pipe. This arrangment traps air in the pump casing and air-locks occur if the well level is drawn too low. Air release valves were installed on the discharge pipes at the pumps but are not automatic and air locks could develop at critical times.

The new pump is a close-coupled unit wherein the motor and pump are separated with the motor mounted over the pump and the shafts connected by a coupling flange.

This station is equipped with a 62.5 KW generator set with automatic transfer switch. There is sufficient stand-by capacity to operate one pump only with ancilliary equipment operating. The station design will not allow the motor to be installed on the grade level floor as that space is occupied by the generator set.

The wet well is approximately 8 feet wide and the bar screen, set at an angle of about 30° from horizontal, covers the complete width of the well. Because of the loss of velocity excessive debris collects on the screen. If the width of flow through the screen was better controlled fine screenings would pass through thereby reducing the amount of screenings to be removed.

A-32

Screenings are raked into buckets, hoisted by hand to grade level and trucked to sanitary landfill.

The wet well does not have mechanical ventilation. There is an open grating approximately 3 feet square covering the entrance which provides some ventilation. An open grating platform covering the entire wet well area is located at bar screen height. Entrance is by means of ladder.

The dry well has mechanical ventilation.

Pump control is by means of bubbler system located in the motor control centre on the main floor.

Recommendations for John Street Pumping Station

The pump header should be raised for automatic air release valves placed on the pump discharges to ensure that air drawn into the pump can be passed through or discharged to prevent air locking the pumps. Since the discharge pipe rises approximately 4 feet before it passes through the station wall it is suggested the pipe header in the station be raised. This would eliminate the air release valves which require maintenance.

The check valve arms and weights are small. They should be checked to ensure adequate counterweight is applied to the valve flaps to ensure closing on shut-down of pumps to prevent back flow and slamming.

Seal water should come from the fresh water supply. This would require a backflow preventer, pressure regulating valve and seal water control feeds to each pump. This seal water system would ensure clean water pressure on the seals before the pump starts up and prevent the initial backflow of sewage into the seals.

When the second pump requires replacement we recommend that a close-coupled unit be considered. The built-together unit has no advantage in this station and has the disadvantage of requiring a special motor with resulting more costly repairs or replacement.

Work should be done on the screen in the wet well to improve the flow condition. We recommend aluminum side plates be installed and the bar screen be placed at an angle of about 45 to 60° so that only the coarse screenings will be retained.

Provision should be made for better ventilation of the wet well and dry well to bring the number of air changes up to the recommendation of the Ministry of the Environment.

The shut-off valves in the suction and discharge piping are Walworth plug valves which appear to have approximately 70% openings. The capacity of the new pump is 900 US gpm or about 2 cfs which means there is a velocity of approximately 10 feet per second in the 6-inch diameter pipe and considerably higher through the plug valves. It would be advantageous to remove these plug valves and install standard gate valves or 100% port opening valves, particularly on the suctions.

Moore Park Pumping Station

Investigation

This station is a Smith and Loveless, pre-fabricated underground station equipped with two 4" x 4" Smith and Loveless pumps rated at 500 US gpm at 40 feet, model 4B2A. The station was installed in June of 1967.

Both suction and discharge piping is 6-inch diameter and the piping configuration is standard for underground pumping stations. The pumps are equipped with John Crane mechanical seals. Filtered pump discharge is used for seal water. It was indicated by the operator that the seals have to be replaced frequently.

A-34

The motors are 7.5 horsepower operating at 1745 rpm. These are built-together units.

There is a stand-by generator set in a separate building above ground to provide temporary stand-by power.

The control panel is equipped with an automatic alternator to switch lead/lag pump condition after each start. Region operations has not had any problems with this control. The pump range in the wet well is 15-inches ranging from a low of 45-inches to a high of 60-inches above the end of the bubbler tube.

Recommendations for Moore Park Pumping Station

If possible, a pure water supply should be provided with a backflow preventer and pressure reducing valve to supply fresh water to the pump seals. Here again the pressure to the seal is not built-up until after the pumps are operating which can cause temporary backflow through the seals before the seal water system is under pressure.

The wet well level could be increased to reduce the number of starts on the equipment if starts become too frequent.

The pumps are built-together type and if being replaced perhaps the close-coupled types should be considered if space is available.

GENERAL RECOMMENDATIONS

Flows into the stations should be reviewed to ensure the pumps have adequate capacity. We suggest the Ministry of the Environment recommendation of one spare pump to provide stand-by capacity equal to that of the larger unit be followed. In two pump stations this would require both pumps to be of the same capacity and that one pump provide the rated capacity requirement of the station.

APPENDIX B

WATER POLLUTION CONTROL PLANT
MONITORING PROGRAM TABLES

APPENDIX 'B'

WATER POLLUTION CONTROL PLANT

MONITORING PROGRAM

TABLES

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GEORGETOWN WPCP

TABLE 5-1
RAW SEWAGE AND PRIMARY EFFLUENT CHARACTERISTICS
DURING CONSTITUENT # 1 (A & B1/6)

003.01

SAMPLE DATES (1980 - 1981)	RAW SEWAGE				PRIMARY EFFLUENT - PLANT A						PRIMARY EFFLUENT - PLANT B					
	pH	S.S. (mg/L)	BOD_5 (mg/L)	P Tot. (mg/L)	pH	S.S. (mg/L)	BOD_5 (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P Tot. (mg/L)	pH	S.S. (mg/L)	BOD_5 (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P Tot. (mg/L)
OCTOBER 08-10	7.9	164	99	4.2	7.8	86	96	14.8	17.1	5.1	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
" 15-16	7.9	149	80	5.9	7.6	114	100	16.9	18.8	6.3	"	"	"	"	"	"
" 23-24	7.5	166	102	6.8	7.6	116	110	15.7	18.4	7.8	"	"	"	"	"	"
" 29-29	7.6	110	67	4.0	7.4	102	84	-	-	4.4	"	"	"	"	"	"
NOVEMBER 06-07	8.3	158	108	4.9	7.9	79	96	-	-	4.2	"	"	"	"	"	"
" 13-14	8.2	153	107	4.2	8.0	80	111	-	-	3.8	"	"	"	"	"	"
" 20-21	7.4	113	139	4.2	7.8	79	87	16.8	27.2	3.2	"	"	"	"	"	"
" 27-28	8.3	188	107	10.6	8.0	94	106	19.7	29.7	10.2	"	"	"	"	"	"
DECEMBER 04-03	7.7	213	102	4.42	7.6	64	110	14.78	20.94	3.84	"	"	"	"	"	"
" 10-11	8.3	203	171	4.24	8.0	85	95	-	-	3.00	8.0	66	90	"	"	4.0
" 18-19	8.0	122	79	4.40	7.8	86	111	-	-	4.40	7.8	61	75	"	"	4.94
" 23-24	7.5	130	110	6.16	7.9	97	101	-	-	7.84	7.7	99	89	"	"	7.44
"	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
JANUARY 07-08	8.1	141	201	4.48	8.1	100	228	19.60	33.10	4.48	8.4	176	216	16.06	35.70	5.78
" "	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"
" 20-21	7.9	106	222	4.16	8.0	111	228	-	-	4.84	7.8	188	230	N.A.	N.A.	4.84
" 28-29	7.8	172	240	4.84	7.9	110	227	-	-	5.04	7.9	72	219	"	"	3.84
FEBRUARY 03-04	7.9	150	220	5.90	8.1	110	170	-	-	5.50	8.3	88	192	"	"	5.04
" 11-12	8.1	125	294	3.60	7.9	102	294	10.08	24.02	4.0	8.0	76	288	7.34	16.52	3.20
AVERAGE	7.9	158	145	5.10	7.9	103	130	16.11	23.79	5.19	8.0	88	170	12.99	25.61	4.82
MAXIMUM	8.3	303	294	10.6	8.3	162	294	19.7	29.7	10.2	8.4	176	288	16.96	35.70	7.44
MINIMUM	7.4	106	79	3.60	7.4	79	84	10.08	17.1	3.2	7.7	88	75	7.34	16.52	3.20

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GEORGETOWN, WPCP

TABLE 1-2
RAW SEWAGE AND PRIMARY EFFLUENT CHARACTERISTICS
DURING CONDITION #2 (0.5 l/s)

603.01

SAMPLE DATES (1981)	RAW SEWAGE				PRIMARY EFFLUENT - PLANT A O/S						PRIMARY EFFLUENT - PLANT B					
	pH	S.S. (mg/L)	COD_t (mg/L)	P Tot. (mg/L)	pH	S.S. (mg/L)	BOD_t (mg/L)	NH_3-N (mg/L)	TSS (mg/L)	P Tot. (mg/L)	pH	S.S. (mg/L)	BOD_t (mg/L)	NH_3-N (mg/L)	TSS (mg/L)	P Tot. (mg/L)
FEBRUARY 17-18	7.5	102	106	4.0							7.6	63	93	-	-	5.2
" 25-26	7.2	139	98	5.04							7.6	91	66	-	-	7.2
MARCH 03-04	8.0	119	156	5.04							7.9	72	99	15.12	16.22	6.16
" 12-13	7.8	129	183	4.04							7.6	61	90	-	-	3.84
" 16-17	7.9	161	114	5.04							8.1	99	90	-	-	4.24
" 25-27	8.1	194	130	5.60							7.9	62	61	-	-	3.84
APRIL 01-02	8.0	178	111	4.04							7.8	60	68	22.62	29.12	3.62
" 09-10	7.7	196	101	4.64							7.6	142	90	-	-	3.44
" 13-14	7.0	102	84	4.80							7.5	98	60	-	-	3.84
" 22-23	7.9	164	132	5.04							7.8	76	90	-	-	3.84
" 28-29	7.9	166	131	6.0							7.8	166	118	-	-	6.0
MAY 05-06	7.9	164	120	4.80							7.5	110	107	-	37.1	5.36
" 14-15	7.6	172	107	5.88							7.6	110	84	-	38.2	5.29
" 20-21	7.6	148	109	4.4							7.7	116	78	-	-	4.4
" 28-29	7.5	202	114	5.28							7.6	126	80	-	-	4.84
JUNE 02-03	8.0	130	100	5.28							7.8	100	72	18.4	24.9	4.84
" 08-09	8.0	298	102	6.60							7.6	76	69	-	-	4.4
" 18-19	8.0	416	156	7.40							7.7	88	69	-	-	2.8
AVERAGE	7.7	181	121	5.30							7.7	97	87	18.71	23.51	4.6
MAXIMUM	8.1	416	156	8.00							8.1	166	118	22.62	38.2	6.4
MINIMUM	7.0	102	84	4.0							7.4	60	60	15.12	16.22	2.80

SEWAGE TREATMENT REPORT

TABLE 1-3
RAW SEWAGE AND PRIMARY EFFLUENT CHARACTERISTICS
DURING CONDITION #2 (A & B1/s)

603.01

SAMPLE DATES (1981)	RAW SEWAGE				PRIMARY EFFLUENT - PLANT A (not sampled)						PRIMARY EFFLUENT - PLANT B					
	pH	S.S. (mg/L)	BOD_5 (mg/L)	P Tot. (mg/L)	pH	S.S. (mg/L)	$CBOD_5$ (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P Tot. (mg/L)	pH	S.S. (mg/L)	BOD_5 (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P Tot. (mg/L)
JUNE 24-25	7.7	240	143	5.60							7.4	31	36	-	-	4.4
" 29-30	7.6	236	102	5.00							7.5	72	42	14.6	23.9	4.2
JULY 08-06	7.7	266	96	5.0							7.4	90	74	-	-	4.8
" 14-16	7.3	170	104	4.5							7.2	120	90	-	-	4.4
" 22-23	7.4	212	104	7.5							7.1	82	61	-	-	6.0
" 28-30	7.3	94	71	3.3							7.1	66	60	-	-	3.2
AUGUST 04-05	7.0	184	139	3.0							7.0	60	70	9.7	19.8	3.6
" 11-12	7.4	184	110	6.6							7.2	60	59	-	-	4.8
" 19-20	7.4	254	168	6.2							7.1	92	67	-	-	5.2
AVERAGE	7.4	198	120	5.04							7.2	79	67	12.1	21.9	4.5
MAXIMUM	7.7	254	164	7.6							7.5	122	90	14.6	23.9	6.0
MINIMUM	7.3	94	71	3.3							7.0	31	36	9.7	19.8	3.2

SEWAGE TREATMENT REPORT

TABLE 1-4
RAW SEWAGE AND PRIMARY EFFLUENT CHARACTERISTICS
DURING CONDITION #4 'B 1/s)

603.01

SAMPLE DATES (1981)	RAW SEWAGE				PRIMARY EFFLUENT - PLANT A (o/s)						PRIMARY EFFLUENT - PLANT B					
	pH	S.S. (mg/L)	BOD_5 (mg/L)	P Tot. (mg/L)	pH	S.S. (mg/L)	BOD_5 (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P Tot. (mg/L)	pH	S.S. (mg/L)	BOD_5 (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P Tot. (mg/L)
AUGUST 04-05	7.4	232	102	7.6							7.2	64	64	14.6	24.3	5.2
SEPTEMBER 02-03	7.3	212	90	4.2							7.3	114	69	-	-	5.2
" 08-09	7.6	160	170	4.0							7.4	92	122	-	-	3.2
" 16-17	7.1	234	206	5.4							6.9	134	130	-	-	4.2
" 22-23	7.2	160	130	10.0							7.0	146	90	-	-	5.6
" 29-30	7.3	280	160	6.8							7.1	160	110	-	-	3.4
OCTOBER 06-07	N.A.	N.A.	N.A.	N.A.							N.A.	N.A.	N.A.	-	-	N.A.
" 13-13	"	"	"	"							"	"	"	"	"	"
" 20-21	"	"	"	"							"	"	"	"	"	"
AVERAGE	7.3	215	153	6.3							7.2	112	107	14.6	24.3	4.5
MAXIMUM	7.6	280	206	10.0							7.4	146	130	-	-	5.6
MINIMUM	7.1	150	90	4.0							6.9	64	64	-	-	3.2

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SEDIMENTATION WPCP

TABLE 2-1
ACTIVATED SLUDGE SYSTEMS : PLANT A & B CONDITIONS
DURING CONDITION #1 (A & B1/s)

803.01

SAMPLE DATES (1980 - 1981)	PLANT A (AVG. OF TWO TANKS)						PLANT B (AVG. OF TWO TANKS)								
	AERATION D.O. (mg/L)	MLSS (mg/L)	SVI (mL/g)	MLVSS (.)	AER. TEMP (C)	SEWAGE FLOW m /d.	AERATION D.O. (mg/L)	MLSS (cg/L)	SVI (mL/g)	MLVSS (:)	AER. TEMP (C)	W.A.S. (Kg/d)	SRT (days)	SEWAGE FLOW m³/d	Fe DOSAGE mg/L)
OCTOBER 09-10	4.0	2225	74	51.3	16	N.A.	2.9	2700	82	-	15	N.A.	N.A.	N.A.	N.A.
" 15-16	4.0	2285	77	50.3	14	"	3.4	2630	82	50.1	14	"	"	"	"
" 23-24	4.9	2850	81	50.4	13.5	"	3.6	2580	70	51.3	13.0	"	"	"	"
" 28-29	4.4	1800	78	49.3	13.5	"	6.0	2155	79	51.9	13.0	"	"	"	"
NOVEMBER 06-07	-	1830	-	52.2	-	"	-	2295	-	52.6	-	"	"	"	"
" 13-14	6.3	2400	-	47.4	13.0	"	5.6	2755	-	52.8	12.0	"	"	"	"
" 20-21	7.3	1680	89	47.3	10.5	"	6.0	2070	87	50.2	11.5	"	"	"	"
" 27-28	7.3	1582	70	51.7	10	4900	6.3	1900	79	56.6	11	"	"	4900	"
DECEMBER 02-03	7.6	2148	63	54.8	11.1	5300	6.0	2944	62	-	12.0	"	"	5300	6.5
" 10-11	8.9	2236	56	54.7	9.8	4800	8.1	2105	86	52.6	9.5	"	"	4800	7.5
" 18-19	9.0	1770	51	50.7	8.0	4050	7.8	2722	79	57.3	8.5	"	"	4050	9.5
" 23-24	9.1	2412	44	52.1	8.0	3850	7.4	2738	82	52.7	8.5	"	"	3850	9.1
															10.4
JANUARY 07-08	6.9	3734	36	46.2	7.0	3918	8.4	2847	88	56.9	6.0	"	"	3918	10.0
"															10.3
" 20-21	9.2	2296	59	58.2	8.0	3942	9.6	2548	88	55.7	6.3	"	"	3942	9.6
" 28-29	10.7	1905	79	54.2	7.0	3940	10.1	2710	107	52.6	7.0	"	"	3940	9.2
FEBRUARY 05-06	7.6	2510	94	57.0	7.0	3749	8.5	2690	91	53.1	6.0	"	"	3749	10.9
" 11-12	8.2	2702	98	56.0	7.0	4066	7.7	3864	69	57.6	6.5	"	"	4066	9.2
AVERAGE	7.2	2248	70	52.1	10.2	4252	6.7	2662	82	53.6	10.0			4252	9.3
MAXIMUM	10.7	3734	98	58.2	16.0	5300	10.1	3944	107	57.6	15			5300	10.9
MINIMUM	4.0	1582	36	46.2	7.0	3749	2.9	1900	62	50.1	6.0			3749	6.5

GEORGETOWN WPCP

TABLE 2-2
ACTIVATED SLUDGE SYSTEMS : PLANT A & B
DURING CONDITION #2 (E 1/s)

003.01

SAMPLE DATES (1981)	PLANT A o/s						PLANT B (AVG. OF TWO TANKS)								
	AERATION D.O. mg/L	MLSS (mg/L)	SVI (mL/g)	MLVSS (-)	AER. TEMP (°C)		AERATION D.O. (mg/L)	MLSS (mg/L)	SVI (mL/g)	MLVSS (%)	AER. TEMP (°C)	M.A.S. (kg/d)	SRT (days)	SLUDGE FLOW m³/d	Fe DOSAGE (mg/L)
FEBRUARY 17-18							10.3	3396	-	55.1	7.5	N.A.	N.A.	16663	5.7
" 25-26							8.4	3608	73	55.1	9.0	"	"	10144	7.6
MARCH 03-04							7.1	3318	83	58.0	8.5	"	"	8748	13.4
" 12-13							6.5	4596	-	57.6	9.0	1213.8	5.2	8702	14.0
" 16-17							4.9	4072	124	57.8	8.5	724.2	8.5	8369	15.8
" 25-27							7.3	5214	68	58.7	10.0	636.9	9.9	8321	14.1
APRIL 01-02							4.2	4542	77	56.5	10.0	2391.6	2.7	7986	15.3
" 09-10							3.0	4620	70	55.4	12.0	2081.5	3.1	7726	21.2
" 13-14							4.5	3740	173	56.7	11.0	2258.5	2.8	9317	21.0
" 22-23							6.1	4042	77	57.7	11.0	2338.0	2.7	9520	12.3
" 28-29							6.4	3758	65	58.1	12.0	1938.7	3.3	7911	11.2
MAY 05-06							6.5	4023	65	59.4	13.0	1881.2	6.3	7614	10.7
" 14-15							7.7	4432	64	58.8	13.0	1840.2	3.5	8037	10.3
" 20-21							5.4	4688	65	59.8	14.5	1404.5	4.6	8536	10.4
" 28-29							6.1	4174	75	60.1	14.5	1735.5	3.7	8852	9.8
JUNE 02-03							8.0	3842	73	57.4	14.5	1310.1	4.8	8756	10.9
" 08-09							5.8	3598	83	56.4	15.0	1543.	4.1	8936	10.9
" 18-19							6.0	2732	73	52.5	16.5	N.A.	N.A.	8812	10.3
AVERAGE							6.3	3989	82	57.3	11.6	1683.2	4.7	9019	12.5
MAXIMUM							10.3	5214	173	60.1	16.5	2391.6	9.9	16663	21.2
MINIMUM							3.0	2732	64	52.5	7.5	636.9	2.7	7614	5.7

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GEORGETOWN WPCP

TABLE 2-3
ACTIVATED SLUDGE SYSTEMS : PLANT A & B
DURING CONDITION #3 (A & B1/s)

603.01

SAMPLE DATES (1981)	PLANT A (not sampled)						PLANT B (AVG. OF TWO TANKS)								
	AERATION D.O. (mg/L)	MLSS (mg/L)	SVI (mL/g)	MLVSS (-)	AER. TEMP (°C)		AERATION D.O. (mg/L)	MLSS (mg/L)	SVI (mL/g)	MLVSS (-)	AER. TEMP (°C)	M.A.S. (kg/d)	SRT (days)	SEWAGE FLOW m³/d.	Fe DOSAGE (mg/L)
JUNE 24-25							6.3	4218	70	54.8	16.0	416.5	14.9	4564	10.8
" 29-30							3.5	3360	61	62.4	17.0	764.8	8.2	6171	9.6
JULY 08-09							3.6	4120	67	50.2	20.0	1190.2	5.3	6439	10.9
" 14-15							3.2	4426	79	50.9	18.5	996.8	6.4	5616	12.5
" 22-23							3.1	3626	65	50.8	20.0	990.0	10.6	5329	12.9
" 29-30							4.3	4184	68	51.4	17.0	661.7	9.5	5319	12.1
AUGUST 04-05							3.9	3896	62	49.7	18.0	1778.1	3.6	6107	12.3
" 11-12							2.9	3320	63	50.8	19.0	1428.0	4.5	5924	11.6
" 19-20							2.8	4278	53	54.0	18.5	1035.1	6.1	4849	15.6
AVERAGE							3.7	3992	65	51.7	18.2	983.7	7.9	5593	12.0
MAXIMUM							6.3	4426	79	54.8	20.0	1778.1	14.9	6439	15.6
MINIMUM							2.8	3320	53	49.7	16.0	416.5	3.6	4584	9.6

GEORGETOWN WPCP

TABLE 2-4
ACTIVATED SLUDGE SYSTEMS : PLANT A & B
DURING CONDITION #4 (A & B1/s)

603.01

SAMPLE DATES	PLANT A (a/s)						PLANT B (AVG. OF TWO TANKS)								
	AERATION D.O. (mg/L)	MLSS (mg/L)	SVI (mL/g)	MLVSS (%)	AER. TEMP (°C)		AERATION D.O. (mg/L)	MLSS (mg/L)	SVI (mL/g)	MLVSS (%)	AER. TEMP (°C)	M.A.S. (kg/d)	SRT (days)	SEWAGE FLOW m³/d	Fe DOSAGE (mg/L)
AUGUST 24-25							2.0	4438	69	52.9	19.0	1470.2	6.1	8188	16.1
SEPTEMBER 02-03							2.2	5240	64	55.2	20	2916.2	3.1	16834	11.9
" 08-09							2.1	4332	N.A.	50.6	18	2997.2	3.0	9915	12.1
" 16-17							1.9	7086	115	80.2	18	3673.7	2.5	9228	13.8
" 22-23							2.2	5488	73	49.9	16.5	2992.4	3.0	8884	12.6
" 29-30							2.7	4946	83	54.1	17.0	0	-	8529	12.7
OCTOBER 06-07							N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
" 12-13							"	"	"	"	"	"	"	"	"
" 20-21							"	"	"	"	"	"	"	"	"
AVERAGE							2.2	5255	81	52.2	18.1	2341.6	3.5	10263	13.2
MAXIMUM							2.7	7086	115	55.2	20	3673.7	6.1	16834	16.1
MINIMUM							1.9	4332	64	49.9	16.5	1470.2	2.5	8188	11.9

GEORGETOWN WPCP

TABLE 3-1
SECONDARY TREATMENT SYSTEMS : EFFLUENT CHARACTERISTICS
DURING CONDITIONS #1 (A & B i/s)

603.01

SAMPLING DATES (1960 - 1961)	PLANT A								PLANT B							
	pH	S.S (mg/L)	BOD_5 tot (mg/L)	BOD_5 sol (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P tot (mg/L)	P sol (mg/L)	pH	S.S. (mg/L)	BOD_5 tot (mg/L)	BOD_5 sol (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P tot (mg/L)	P sol (mg/L)
OCTOBER 09-10	N.A.	48	23	7	<0.1	N.A.	2.2	1.7	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
" 15-16	"	14	20	12	<0.1	"	2.1	1.8	"	"	"	"	<0.1	"	"	"
" 23-24	"	22	14	6	<0.1	"	1.5	1.3	"	"	"	"	<0.1	"	"	"
" 28-29	7.7	13	12	4	<0.1	3.4	1.7	1.4	"	"	"	"	<0.1	"	"	"
NOVEMBER 06-07	8.1	18	4	1	0.5	2.7	0.9	0.7	"	"	"	"	0.34	"	"	"
" 13-14	8.0	16	10	2	0.91	4.54	2.2	1.3	"	"	"	"	0.39	"	"	"
" 20-21	7.6	39	25	3	0.84	4.1	0.9	0.5	"	"	"	"	0.34	"	"	"
" 27-28	8.0	20	18	3	0.90	2.6	1.5	1.0	"	"	"	"	0.34	"	"	"
DECEMBER 02-03	7.9	15	14	2	0.34	3.3	0.9	0.5	"	"	"	"	4.87	"	"	"
" 10-11	7.9	7	12	7	0.95	2.46	0.38	0.10	8.1	5.0	10	7.0	0.62	3.42	0.68	0.64
" 18-19	7.5	7	5	2	0.56	2.69	0.32	0.10	7.7	5.0	6.0	1.0	0.87	3.30	0.96	0.76
" 23-24	7.7	6	15	6	1.12	4.65	1.42	0.90	7.6	5.0	15.0	6.0	0.45	2.80	2.28	1.96
JANUARY 07-08	7.9	7	20	8	0.73	3.30	1.16	0.96	7.9	7.0	23	10.0	0.45	0.90	0.90	0.88
" 20-21	7.7	13	36	10	2.46	6.22	0.90	0.50	7.7	8.0	43	15	2.74	6.44	0.90	0.44
" 28-29	7.7	17	38	13	0.29	4.89	0.96	0.52	7.8	5.0	27	13	0.36	3.92	0.64	0.52
FEBRUARY 05-06	7.9	23	25	8	0.56	3.92	0.86	0.44	8.2	8.0	12	4	0.67	3.30	0.70	0.56
" 11-12	7.5	42	40	22	1.68	3.14	0.86	0.38	7.4	21	37	14	2.74	5.38	0.76	0.44
AVERAGE	7.8	19	20	7	0.72	3.71	1.21	0.83	7.8	8.0	22	8.8	31.47	3.68	0.97	0.86
MAXIMUM	8.1	48	40	22	2.46	6.22	2.2	1.8	8.2	21	43	15	4.87	6.44	2.28	1.96
MINIMUM	7.5	6	4	1	<0.1	2.6	0.32	0.10	7.4	5.0	6.0	1.0	<0.1	0.90	0.64	0.44

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TABLE 3-2
SECONDARY TREATMENT SYSTEMS : EFFLUENT CHARACTERISTICS
DURING CONDITION +Z (8 1/s)

603.01

SAMPLING DATES (1981)	PLANT A o/s								PLANT B							
	pH	S.S (mg/L)	BOD_5 tot (mg/L)	BOD_5 sol (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P tot (mg/L)	P sol (mg/L)	pH	S.S. (mg/L)	BOD_5 tot (mg/L)	COD_5 sol (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P tot (mg/L)	P sol (mg/L
FEBRUARY 17-18									7.5	21	38	8	2.24	4.90	1.22	0.56
" 25-26									7.5	12	26	7	5.43	8.51	1.58	1.48
MARCH 03-04									7.6	8.0	13.0	1.0	12.88	16.52	1.54	1.54
" 12-13									7.6	9.0	14.0	3.0	15.68	18.87	0.50	0.22
" 16-17									7.7	3.0	9.0	2.0	13.38	16.13	0.32	0.20
" 25-27									7.9	7.0	7.0	1.0	22.90	23.30	0.30	0.20
APRIL 01-02									7.7	9.0	17.0	3.0	16.35	18.98	0.38	0.26
" 08-10									7.4	9.0	12.0	3.0	9.52	12.54	0.24	0.12
" 13-14									7.4	3.0	6.0	2.0	7.80	8.70	0.24	0.11
" 22-23									7.2	10.0	43.0	2.0	5.90	9.30	0.48	0.10
" 28-29									7.6	6.0	16.0	2.0	8.8	12.0	1.9	1.74
MAY 05-06									7.8	7.0	8.0	4.0	14.5	17.7	0.74	0.64
" 14-15									7.8	6.0	4.0	2.0	17.4	20.2	1.32	1.16
" 20-21									7.8	17.0	2.0	1.0	17.5	18.8	0.66	0.58
" 28-29									7.6	11.0	8.0	1.0	15.9	19.1	1.06	0.86
JUNE 02-03									7.7	6.0	6.0	1.0	18.4	24.9	2.10	1.96
" 08-09									7.7	5.0	6.0	2.0	19.0	21.7	0.76	0.64
" 16-19									7.9	3.0	5.0	2.0	15.0	18.7	0.75	0.70
AVERAGE									7.6	8.4	13.3	2.6	13.25	14.75	0.91	0.73
MAXIMUM									7.9	21	43.0	8	22.90	24.9	2.10	1.96
MINIMUM									7.2	3.0	2.0	1.0	2.24	4.90	0.24	0.10

GEORGETOWN WPCP

TABLE 3-3
SECONDARY TREATMENT SYSTEMS : EFFLUENT CHARACTERISTICS
DURING CONDITION #3 (A & B 1/s)

803.01

SAMPLING DATES (1981)	PLANT A (1/s)								PLANT B (1/s)							
	pH	S.S (mg/L)	BOD_5 tot (mg/L)	BOD_5 sol (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P tot (mg/L)	P sol (mg/L)	pH	S.S. (mg/L)	BOD_5 tot (mg/L)	BOD_5 sol (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P tot (mg/L)	P sol (mg/L)
JUNE 24-25	N.A.	N.A.	N.A.	N.A.	<0.1	N.A.	N.A.	N.A.	7.3	5.0	3.0	2.0	0.1	1.96	0.95	0.65
" 29-30	"	"	"	"	<0.1	"	"	"	7.4	6.0	7.0	4.0	0.1	2.9	1.0	0.9
JULY 08-09	"	"	"	"	<0.1	"	"	"	7.5	3.0	8.0	2.0	<0.1	3.1	1.05	0.9
" 14-15	"	"	"	"	<0.1	"	"	"	7.3	4.0	11.0	5.0	<0.1	3.1	0.95	0.80
" 22-23	"	"	"	"	<0.1	"	"	"	7.1	6.0	8.0	2.0	<0.1	2.7	1.45	1.35
" 29-30	"	"	"	"	<0.1	"	"	"	7.1	2.0	6.0	2.0	<0.1	1.7	0.30	0.20
AUGUST 04-05	"	"	"	"	<0.1	"	"	"	6.9	4.0	11.0	3.0	0.4	3.9	0.5	0.3
" 11-12	"	"	"	"	<0.1	"	"	"	7.2	6.0	8.0	2.0	0.5	4.5	1.45	1.15
" 19-20	"	"	"	"	<0.1	"	"	"	7.2	6.0	9	3	1.7	4.8	0.75	0.65
AVERAGE					<0.1				7.2	5.3	7.9	2.8	0.4	3.18	0.93	0.76
MAXIMUM					<0.1				7.5	8.0	11.0	5.0	1.7	4.8	1.45	1.35
MINIMUM					<0.1				6.9	2.0	3.0	2.0	<0.1	1.7	0.30	0.20

GEORGETOWN WPCP

TABLE 3-4
SECONDARY TREATMENT SYSTEMS : EFFLUENT CHARACTERISTICS
DURING CONDITION #4 (B 1/s)

803.01

SAMPLING DATES (1981)	PLANT A (o/s)								PLANT B							
	pH	S.S (mg/L)	BOD_5 tot (mg/L)	BOD_5 sol (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P tot (mg/L)	P sol (mg/L)	pH	S.S. (mg/L)	BOD_5 tot (mg/L)	BOD_5 sol (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P tot (mg/L)	P sol (mg/L)
AUGUST 24-25									7.3	6	8	2	3.0	4.3	1.25	1.10
SEPTEMBER 02-03									7.2	7.0	9	2	3.8	7.3	0.70	0.50
" 08-09									7.5	3	19.5	6.5	5.0	6.3	0.45	0.35
" 16-17									6.8	7	15	7	4.1	7.6	0.55	0.25
" 22-23									6.9	14	6	2	2.7	6.3	0.9	0.4
" 29-30									7.0	15	19	6	2.6	7.1	0.7	0.15
OCTOBER 06-07																
" 12-13																
" 20-21																
AVERAGE									7.1	9	15.3	4.3	3.5	6.5	0.76	0.46
MAXIMUM									7.5	15	19.5	7	5.0	7.6	1.25	1.10
MINIMUM									6.8	3	6	2	2.6	4.3	0.45	0.15

B-10

603.01

GEORGETOWN WPCP

TABLE 4-1
CHLORINE CONTACT CHAMBER EFFLUENT CHARACTERISTICS
DURING CONDITION #1 (A & B 1/s)

SAMPLE DATES (1980-81)	COMBINED FINAL EFFLUENT : PLANTS A & B IN SERVICE								
	pH	S.S (mg/L)	BOD_5 tot (mg/L)	BOD_5 sol (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P tot (mg/L)	P sol (mg/L)	
OCTOBER 09-10	7.7	1	5	4	<0.1	5.8	1.5	1.3	
" 15-16	7.6	6	2.3	2.1	<0.1	5.2	1.4	1.35	
" 23-24	7.6	2.5	5.0	3.0	<0.1	4.5	1.3	1.1	
" 28-29	7.5	2.0	11.0	7.0	<0.1	2.2	0.96	0.9	
NOVEMBER 06-07	7.7	5.0	3.0	1.0	0.34	1.51	0.64	0.5	
" 13-14	7.6	1.0	4.0	<1.0	0.22	1.29	0.9	0.7	
" 20-21	7.5	2.0	2.0	1.0	0.60	2.58	0.5	0.4	
" 27-28	7.7	1.0	2.0	<1.0	0.78	6.1	2.1	2.0	
DECEMBER 02-03	7.5	3.0	1.0	<1.0	3.92	5.66	0.9	0.86	
" 10-11	7.8	1.0	<1.0	<1.0	1.23	3.19	0.42	0.38	
" 18-19	7.6	1.0	2.0	1.0	1.46	3.3	0.50	0.20	
" 23-24	7.25	1.0	4.0	2.0	0.22	2.58	2.06	1.64	
JANUARY 07-08	7.7	3.0	5.0	5.0	1.06	3.14	0.86	0.80	
" 20-21	7.5	3.0	2.0	<1.0	1.23	3.92	0.52	0.50	
" 28-29	7.4	3.0	9.0	4.0	0.39	3.53	0.64	0.52	
FEBRUARY 05-06	7.7	4.0	2.0	<1.0	0.62	3.02	0.50	0.38	
" 11-12	7.0	5.0	13	4.0	1.34	3.14	0.44	0.38	
AVERAGE	7.6	2.6	4.3	2.4	0.83	3.57	0.99	0.81	
MAXIMUM	7.8	6	13	7.0	3.92	6.1	2.1	2.0	
MINIMUM	7.0	1.0	<1.0	<1.0	<0.1	1.29	0.42	0.20	

GEORGETOWN WPCP

TABLE 4-2
CHLORINE CONTACT CHAMBER EFFLUENT CHARACTERISTICS
DURING CONDITION #2 (B 1/s)

603.01

SAMPLE DATE (1981)	COMBINED FINAL EFFLUENT: PLANT B ONLY IN SERVICE								
	pH	S.S. (mg/L)	BCD_5 tot (mg/L)	BOD_5 sd (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P tot (mg/L)	P sd (mg/L)	
FEBRUARY 17-18	6.8	2.0	5	4.0	1.57	3.30	0.50	0.44	
" 25-26	7.6	3.0	6	3.0	3.42	6.22	1.48	1.48	
MARCH 03-04	7.2	3.0	1.0	<1.0	12.21	15.96	1.54	1.48	
" 12-13	7.8	4.0	1.0	<1.0	13.44	18.76	0.32	0.22	
" 16-17	8.0	2.0	2.0	1.0	13.27	16.02	0.26	0.20	
" 26-27	7.9	4.0	1.0	<1.0	22.51	22.68	0.25	0.20	
APRIL 01-02	7.5	4.0	2.0	1.0	16.18	19.82	0.32	0.26	
" 09-10	7.3	7.0	3.0	1.0	6.78	11.54	0.22	0.12	
" 13-14	7.0	2.0	2.0	<1.0	7.50	7.80	0.22	0.11	
" 22-23	6.8	2.0	3.0	1.0	5.80	8.30	0.22	0.10	
" 28-29	7.8	3.0	4.0	2.0	7.60	11.6	1.80	1.70	
MAY 05-06	7.8	6.0	6.0	4.0	14.2	17.7	0.74	0.64	
" 14-15	7.9	2.0	2.0	1.0	16.5	16.9	1.22	1.16	
" 20-21	7.8	3.0	3.0	<1.0	17.4	17.4	0.80	0.58	
" 28-29	7.6	6.0	6.0	1.0	15.7	18.5	1.0	0.86	
JUNE 02-03	7.7	7.0	7.0	1.0	17.2	20.1	2.20	1.96	
" 08-09	7.8	5.0	5.0	1.0	18.3	20.5	0.76	0.64	
" 18-19	8.0	7.0	7.0	2.0	15.1	19.2	0.80	0.70	
AVERAGE	7.6	4.0	4.0	2.0	12.48	15.13	0.81	0.71	
MAXIMUM	8.0	7.0	7.0	4.0	22.51	22.68	2.20	1.70	
MINIMUM	6.8	2.0	1.0	<1.0	1.57	3.30	0.22	0.10	

B-12

603.01

GEORGETOWN WPCP

TABLE 4-3
CHLORINE CONTACT EFFLUENT CHARACTERISTICS
DURING CONDITION #3 (A & B 1/s)

SAMPLE DATES (1981)	COMBINED FINAL EFFLUENT : PLANTS A & B IN SERVICE								
	pH	S.S (mg/L)	BOD_5 tot (mg/L)	BOD_5 sol (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P tot (mg/L)	P sol (mg/L)	
JUNE 24-25	7.2	3.0	3.0	1.0	<0.1	1.74	0.90	0.85	
" 29-30	7.4	5.0	5.0	4.0	<0.1	2.0	0.95	0.9	
JULY 06-09	7.3	4.0	3.0	2.0	<0.1	2.3	0.95	0.9	
" 14-15	7.7	2.0	5.0	3.0	<0.1	2.0	0.85	0.80	
" 22-23	7.2	4.0	1.0	<1.0	<0.1	2.3	1.40	1.35	
" 29-30	7.2	2.0	1.0	1.0	<0.1	1.6	0.25	0.20	
AUGUST 04-05	6.7	4.0	5.0	3.6	<0.1	3.4	0.35	0.30	
" 11-12	7.3	2.0	N.A.	2.0	<0.1	3.1	1.20	1.15	
" 19-20	7.3	3	3	1.0	0.5	3.7	0.7	0.55	
AVERAGE	7.3	3.0	3.0	2.1	0.1	2.46	0.84	0.78	
MAXIMUM	7.7	5.0	5.0	4.0	0.5	3.7	1.40	1.35	
MINIMUM	6.7	2.0	1.0	<0.1	<0.1	1.6	0.25	0.20	

GEORGETOWN WPCP

TABLE 4-4
CHLORINE CONTACT CHAMBER CHARACTERISTICS
DURING CONDITION #4 (B 1/s)

603.01

SAMPLE DATES (1981)									
	pH	S.S (mg/L)	BOD_5tot (mg/L)	BOD_5sol (mg/L)	NH_3-N (mg/L)	TKN (mg/L)	P tot (mg/L)	P sol (mg/L)	
AUGUST 24-25	7.5	4	1	<1	1.3	3.2	1.2	1.1	
SEPTEMBER 2-03	7.3	6	1	<1	3.6	6.5	0.55	0.50	
" 08-09	7.4	13	3.0	3.0	4.9	7.8	0.60	0.35	
" 16-17	6.8	6	5	3	3.9	6.9	0.40	0.25	
" 22-23	6.8	1	4	1	2.2	5.0	0.60	0.4	
" 29-30	6.9	3	6	3	2.5	6.3	0.20	0.15	
OCTOBER 06-07	*N.B.								
" 12-13									
" 20-21									
AVERAGE	7.1	6	4	2	3.1	6.0	0.59	0.46	
MAXIMUM	7.5	13	6	3	4.9	7.8	1.2	1.1	
MINIMUM	6.8	1	1	<1	1.3	3.2	0.20	0.15	
	*N.B. - PLANT A RESTARED ON OCT.1-FINAL EFFLUENT RESULTS ARE AS FOLLOWS:								
A & B 1/s	pH	S.S.	BOD_5tot	BOD_5sol	NH_3-N	TKN	P tot	P sol	
OCTOBER 06-07	7.1	3	2	<1	<0.1	1.9	0.40	0.25	
" 12-13	6.9	2	5	1	<0.1	1.9	0.35	0.25	
" 20-21	7.2	3	4	1	<0.1	4.5	0.25	0.2	

RECORDED FLOWS

at the

GEORGETOWN WATER POLLUTION CONTROL PLANT

TABLE 5-1

DATE 1977	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC
1	2.4	2.7	2.6	3.0	2.9	2.9	2.0	2.2	2.5	3.8	3.6	3.8
2	2.6	3.0	2.7	3.4	2.8	2.8	2.0	2.1	2.1	3.3	2.7	3.0
3	2.8	2.5	2.7	3.0	2.9	2.7	2.0	2.2	2.1	3.1	2.7	2.7
4	2.8	2.6	3.6	3.2	2.6	2.6	2.4	1.8	2.0	3.0	2.6	3.8
5	2.7	3.2	3.0	3.5	2.7	2.8	2.6	2.6	2.2	2.5	2.6	2.9
6	2.8	2.6	2.9	3.3	2.8	3.2	2.9	3.2	2.5	2.3	2.8	2.8
7	2.7	2.6	2.9	3.2	2.5	3.0	2.9	2.9	2.3	1.6	2.8	2.8
8	2.5	2.8	3.3	3.0	2.6	2.9	2.4	3.0	2.4	3.2	3.2	2.8
9	2.6	2.7	2.8	2.9	2.8	2.7	2.1	2.7	2.4	2.7	2.7	2.5
10	2.6	2.5	3.4	2.9	2.8	2.6	2.2	2.5	2.1	2.5	2.8	2.5
11	2.5	2.0	3.3	3.3	2.7	2.6	2.4	2.4	2.2	2.5	2.9	2.7
12	2.6	N.A.	3.7	3.1	2.7	2.9	2.3	2.2	2.6	2.3	3.3	2.7
13	2.6	N.A.	3.6	3.1	2.6	2.8	2.3	2.3	2.9	2.2	2.7	3.5
14	2.5	N.A.	3.3	3.0	2.5	2.9	2.3	2.2	2.5	2.4	3.0	4.3
15	2.5	N.A.	3.3	2.9	2.6	3.1	2.7	2.0	2.4	2.2	2.9	3.5
16	2.5	N.A.	3.2	2.8	2.7	2.9	2.2	2.9	2.8	2.2	2.7	3.2
17	2.7	N.A.	2.9	2.8	2.8	3.0	2.1	2.5	2.6	2.6	2.7	3.0
18	2.7	2.5	3.3	2.8	2.8	3.5	2.4	2.4	2.6	2.3	2.7	3.5
19	2.5	2.6	2.8	2.9	2.8	2.9	2.3	2.2	2.8	2.2	2.5	13.8
20	2.4	2.5	2.8	2.8	2.6	2.5	2.3	2.2	2.7	2.2	2.9	3.7
21	2.2	2.7	3.1	2.9	2.4	2.8	2.2	2.5	2.6	2.0	3.4	3.5
22	2.2	2.8	3.1	3.5	2.3	2.8	2.2	2.3	2.6	1.9	2.9	3.3
23	2.3	2.6	3.2	2.9	2.6	2.8	2.0	2.5	2.5	2.0	2.9	2.8
24	2.3	2.6	3.0	3.4	3.0	2.8	2.4	2.4	3.6	3.0	3.8	3.4
25	2.4	2.6	2.9	3.3	1.8	3.0	2.5	2.3	3.4	2.7	2.3	3.0
26	2.3	2.5	2.8	3.2	2.7	2.9	2.4	2.3	3.6	2.2	2.1	2.8
27	2.3	3.0	3.0	3.0	2.6	3.1	2.7	2.2	3.2	2.3	2.2	2.8
28	2.2	2.7	2.3	3.1	2.6	2.8	2.2	2.4	2.9	2.1	2.3	1.2
29	2.2	--	2.8	3.0	2.7	2.8	2.0	2.5	2.8	2.0	2.3	N.A.
30	2.5	--	3.0	2.9	3.0	2.6	1.9	2.4	2.9	2.1	1.5	N.A.
31	2.7	--	3.2	--	3.0	--	2.1	2.4	--	3.2	--	N.A.
AVE	2.5	2.5	2.5	2.6	2.2	2.4	2.1	2.4	2.6	2.5	2.7	3.1

NOTES: 1. Quantities are expressed in MIGD.

2. Annual average daily flow = 11.36 x 1000 m^3/day
= 2.5 MIGD

NB Accuracy of measuring equipment questionable.

3. Average equivalent flow per Capita = 144.5 gal./day

(Discount accuracy due to dependability of flow meter)

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RECORDED FLOWS

at the

GEORGETOWN WATER POLLUTION CONTROL PLANT

TABLE 5-2

DATE 1978	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC
1	N.A.	N.A.	4.1	12.4	5.3	3.1	7.9	10.6	8.2	7.8	9.1	9.
2	*2.2	N.A.	4.0	11.3	5.5	2.5	9.9	10.1	6.8	7.1	9.1	6.8
3	*3.4	N.A.	3.0	10.7	5.0	2.5	10.4	9.1	5.5	10.5	9.1	11. 3
4	*3.3	N.A.	2.2	9.9	5.1	3.7	10.2	8.0	9.1	7.8	9.1	9.
5	*3.3	N.A.	3.7	13.2	4.8	3.0	10.5	7.2	9.1	8.3	9.1	9.
6	12.1	N.A.	3.6	13.2	4.5	4.1	9.7	8.7	9.1	6.0	9.1	9.
7	13.1	N.A.	4.8	10.3	7.9	3.2	8.5	10.3	10.1	3.0	9.1	12 5
8	10.5	N.A.	4.7	10.3	6.9	2.6	8.4	10.1	10.4	4.6	9.1	11. 3
9	13.7	N.A.	4.5	11.4	5.5	1.8	10.0	10.8	11.8	7.6	9.1	11.3
10	10.9	7.0	3.4	15.5	6.3	1.4	9.9	10.4	2.6	9.2	6.8	9.
11	11.4	1.6	4.4	12.0	6.4	16.0	9.6	9.3	18.2	9.9	5.4	10 5
12	11.4	N.A.	9.2	10.8	12.3	11.8	10.0	9.1	10.6	10.1	11.4	11.3
13	9.1	4.6	14.6	12.4	11.6	12.1	9.7	8.4	10.8	8.6	9.1	9.
14	7.7	6.6	9.3	7.9	N.A.	11.8	8.8	11.4	15.5	8.6	9.1	9.
15	8.2	6.0	N.A.	8.5	7.3	11.4	8.2	11.4	15.6	9.1	9.1	10.6
16	N.A.	5.4	N.A.	9.2	7.9	11.7	9.7	9.0	13.0	9.1	9.1	9.
17	N.A.	4.8	N.A.	8.1	8.6	13.3	9.5	11.4	12.8	9.1	13.6	11 3
18	7.6	4.2	N.A.	9.4	5.7	13.0	9.9	11.4	16.9	9.1	9.1	9.
19	7.7	4.7	N.A.	14.2	7.0	13.4	10.0	11.4	12.9	9.1	9.1	9.1
20	7.1	5.8	N.A.	10.9	5.2	13.2	10.1	11.4	14.1	8.7	9.1	8.
21	6.6	6.0	N.A.	17.3	7.4	12.0	9.6	11.4	13.8	8.7	9.1	8.
22	6.7	5.7	16.3	9.6	5.7	12.9	8.5	11.4	10.1	6.8	9.1	7.
23	4.5	5.2	12.6	9.4	5.1	11.0	9.2	11.4	6.1	9.1	13.5	7.
24	6.8	4.6	10.9	7.4	5.2	11.4	9.0	10.5	6.2	9.1	9.1	7.
25	12.1	4.2	11.2	8.4	3.6	14.3	9.8	9.1	7.3	9.1	8.0	7.
26	11.2	4.3	14.1	7.3	2.7	11.3	9.6	8.6	7.8	8.1	8.0	7.5
27	8.7	8.3	12.2	7.4	2.3	11.1	5.6	9.1	9.3	7.2	9.1	7.
28	8.1	2.5	11.5	5.3	4.2	10.7	4.4	13.6	6.4	6.8	9.1	7.
29	7.9	--	11.6	7.2	5.2	8.8	4.6	9.1	6.2	8.1	9.1	7.5
30	8.5	--	12.9	5.7	4.8	8.1	2.8	9.1	10.0	9.1	9.1	7.
31	8.1	--	13.8	--	3.5	--	9.0	9.1	--	9.1	--	10 3
AVE	9.2	5.0	12.3	10.2	5.6	12.0	8.8	9.0	10.5	8.2	9.2	9.

NOTES: 1. Values marked * not included in Average.

2. Quantities are expressed in 1000 m^3/day.

3. Annual average daily flow = 9.9 x 1000^3/day.
= 2.18 MIGD

4. Average equivalent flow per capita = 126 gal./day

RECORDED FLOWS

at the

GEORGETOWN WATER POLLUTION CONTROL PLANT

TABLE 5-3

DATE 1979	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC
1	12.6	7.7	10.0	12.0	10.4	11.1	8.9	13.2	9.6	11.1	10.9	9.9
2	10.8	N.A.	9.9	16.8	10.5	10.1	9.1	11.0	8.9	10.4	9.5	9.8
3	9.6	7.6	11.8	12.8	12.9	9.9	11.0	9.4	8.3	10.2	8.6	10.7
4	9.3	7.6	20.2	11.8	12.1	10.7	11.0	8.6	11.0	12.5	8.8	10.6
5	8.3	8.3	14.2	17.1	10.3	11.8	10.8	8.2	10.6	10.3	9.9	11.0
6	7.5	8.5	11.2	14.8	11.8	10.8	9.4	8.8	10.9	8.7	10.5	10.9
7	8.0	8.9	12.3	13.3	12.1	11.9	8.9	9.8	10.2	10.2	9.8	10.8
8	8.5	8.8	12.0	12.1	9.6	11.3	8.6	9.7	9.1	11.4	9.6	9.8
9	8.8	7.8	11.8	12.4	11.3	9.8	8.0	10.0	8.9	12.5	13.1	9.7
10	7.7	7.7	11.2	13.4	11.3	10.9	10.0	10.0	10.6	10.8	10.1	10.9
11	8.0	7.4	9.7	13.7	10.5	11.0	11.7	8.6	10.3	11.5	9.5	10.8
12	6.7	8.6	10.0	14.5	13.4	10.6	10.5	8.3	10.4	11.3	10.6	10.6
13	7.7	8.3	16.9	25.1	10.7	10.6	10.2	10.1	12.7	10.6	10.2	10.5
14	8.2	8.1	12.9	24.3	13.2	10.7	9.2	9.8	15.1	9.5	10.1	10.0
15	8.2	8.3	10.4	15.4	17.9	10.4	9.1	9.6	9.9	10.6	10.2	9.3
16	8.2	7.9	9.9	14.0	13.5	9.3	10.6	10.1	9.7	9.9	10.1	9.1
17	7.3	7.5	9.9	12.7	12.4	9.2	10.0	9.5	11.0	10.4	8.9	10.2
18	8.2	7.9	9.9	11.9	9.6	10.7	10.1	8.5	10.7	10.5	8.9	10.0
19	7.7	8.9	11.1	11.7	10.1	11.0	9.7	8.5	10.5	10.5	10.5	9.7
20	7.2	8.4	10.8	11.2	9.1	10.3	8.1	10.3	11.6	9.6	9.8	9.9
21	7.3	10.0	11.4	10.7	9.9	10.8	8.3	12.1	9.8	9.3	9.9	9.5
22	8.3	8.8	11.2	10.8	11.3	10.3	8.4	10.0	9.2	11.1	11.8	13.9
23	8.5	13.3	12.4	11.1	10.8	8.8	9.6	16.5	9.0	10.5	13.9	29.9
24	8.8	10.7	15.6	10.7	11.0	8.9	9.4	11.7	10.2	10.2	11.5	15.5
25	9.0	8.5	13.7	11.0	14.6	10.8	9.2	9.4	10.1	9.9	14.2	24.3
26	9.6	8.7	12.0	13.1	11.3	10.0	9.2	9.4	10.2	9.4	15.8	13.8
27	9.8	10.1	10.5	11.2	11.6	10.5	9.3	10.7	10.5	8.7	13.5	12.3
28	9.1	10.7	10.8	9.7	12.7	10.5	9.6	10.7	10.3	9.3	13.3	11.4
29	9.7	--	12.8	9.9	13.3	10.4	8.0	10.7	9.1	9.9	11.9	10.5
30	7.3	--	16.3	11.1	12.3	10.7	9.3	10.9	9.3	9.6	10.9	10.0
31	8.2	--	13.2	--	12.0	--	12.2	10.2	--	9.8	--	10.1
AVE	8.5	8.4	12.1	13.3	11.7	10.8	10.6	10.1	10.3	10.3	10.9	11.4

NOTES: 1. Quantities are express in $1000m^3$/day

2. Annual Average daily flow = 10.7 x $1000m^3$/day
= 2.35 MIGD

3. Average equivalent flow per Capita = 135.8 gal./day

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RECORDED FLOWS

at the

GEORGETOWN WATER POLLUTION CONTROL PLANT

TABLE 5-4

DATE 1980	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DE
1	9.2	N.A.	7.7	11.2	12.3	9.3	10.7	8.9	8.6	N.A.	N.A.	10.3
2	11.0	8.9	7.8	15.3	12.0	9.4	10.5	7.9	9.5	N.A.	N.A.	10.9
3	10.5	8.1	9.3	13.6	10.7	10.0	10.7	6.9	N.A.	N.A.	N.A.	10.6
4	10.0	8.4	9.1	11.6	10.3	10.4	10.3	6.8	N.A.	N.A.	N.A.	12.3
5	9.3	9.3	9.2	9.7	11.4	10.4	9.1	7.4	N.A.	N.A.	N.A.	9.6
6	9.4	9.2	9.2	11.9	11.4	10.9	8.7	7.6	N.A.	N.A.	N.A.	9.1
7	10.0	9.3	8.6	11.2	10.7	10.2	11.1	7.4	N.A.	N.A.	N.A.	11.1
8	9.6	9.1	9.9	N.A.	10.6	9.6	11.0	7.2	N.A.	N.A.	N.A.	12.9
9	9.7	9.6	8.1	11.5	10.2	11.1	10.3	6.7	N.A.	N.A.	N.A.	10.9
10	9.6	8.1	11.5	11.3	9.6	10.1	10.5	6.0	N.A.	N.A.	N.A.	10.3
11	12.2	8.4	9.3	11.2	9.6	10.0	10.0	7.8	N.A.	N.A.	N.A.	9.6
12	9.8	9.4	9.2	11.2	10.7	10.2	9.0	7.6	N.A.	N.A.	N.A.	9.4
13	9.7	9.4	9.1	10.2	13.1	9.5	8.7	7.7	N.A.	N.A.	N.A.	8.4
14	10.5	9.3	8.9	10.3	10.8	9.0	10.3	7.9	N.A.	N.A.	N.A.	8.3
15	9.2	9.1	8.0	16.5	10.1	9.2	11.7	7.6	N.A.	N.A.	N.A.	9.1
16	9.9	8.8	8.0	8.6	11.6	10.2	10.4	6.7	N.A.	N.A.	N.A.	8.4
17	10.5	8.0	12.0	12.7	11.0	10.0	10.6	6.1	N.A.	N.A.	N.A.	8.7
18	9.8	8.2	9.9	11.6	10.4	10.2	9.9	7.5	N.A.	N.A.	N.A.	8.8
19	8.9	9.4	11.2	9.6	11.2	12.3	8.6	7.5	N.A.	N.A.	N.A.	8.4
20	8.9	9.2	15.3	10.4	11.1	10.5	8.9	7.6	N.A.	N.A.	N.A.	7.1
21	9.8	9.4	22.2	11.3	11.4	9.2	9.9	7.8	N.A.	N.A.	N.A.	8.0
22	9.7	9.3	10.0	11.5	10.6	9.1	13.5	8.9	N.A.	N.A.	N.A.	8.1
23	9.5	9.4	9.5	10.9	9.8	10.4	10.0	6.9	N.A.	N.A.	N.A.	8.1
24	9.5	9.2	11.0	10.9	9.6	10.5	9.6	6.6	N.A.	N.A.	N.A.	7.7
25	9.2	9.6	10.8	10.4	10.9	10.1	9.8	8.3	N.A.	N.A.	N.A.	6.4
26	8.4	9.4	10.6	8.9	11.3	13.0	8.6	8.4	N.A.	N.A.	6.4	7.1
27	8.5	8.9	10.9	19.4	10.3	10.4	9.7	8.4	N.A.	N.A.	8.7	9.2
28	9.8	8.4	10.2	10.6	10.5	9.9	21.3	8.7	N.A.	N.A.	9.8	9.5
29	8.4	8.6	9.7	10.7	10.4	11.3	12.4	8.0	N.A.	N.A.	8.4	8.1
30	9.0	--	9.7	13.4	10.2	9.7	10.3	7.8	N.A.	N.A.	8.8	7.1
31	9.4	--	12.5	--	8.4	--	9.7	11.1	N.A.	N.A.	N.A.	8.5
AVE	9.6	9.0	10.3	11.3	10.7	10.2	10.5	7.9	8.9	N.A.	8.4	9

NOTES: 1. Quantities are expressed in $1000m^3$/day

2. Annual Average daily flow = 9.7 x $1000m^3$/day
= 2.13 MIGD

3. Average equivalent flow per Capita = 123 gal./day

RECORDED FLOWS

at the

GEORGETOWN WATER POLLUTION CONTROL PLANT

TABLE 5-5

DATE 1981	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC
1	6.3	8.0	8.3	8.0	7.2	8.6	9.1	7.7	9.7			
2	6.8	8.0	9.6	8.0	6.8	8.6	9.2	6.9	12.5			
3	6.8	7.6	9.1	7.8	6.9	8.8	9.0	8.6	16.8			
4	7.5	7.4	8.8	7.8	7.9	9.0	12.4	14.4	10.6			
5	8.1	7.4	8.6	7.4	7.9	8.8	12.6	9.1	15.1			
6	7.8	7.5	8.5	7.9	7.6	7.7	10.8	9.0	10.4			
7	7.8	7.0	7.8	7.8	7.7	7.8	9.8	8.3	10.2			
8	7.8	7.2	8.0	7.6	7.6	9.2	9.7	9.2	11.4			
9	7.3	7.8	9.0	7.8	7.0	8.9	9.4	8.0	9.9			
10	7.1	12.6	8.9	7.7	10.1	9.2	9.0	9.4	10.1			
11	7.0	13.7	8.6	7.3	10.2	9.0	7.8	9.7	9.7			
12	7.9	8.1	8.8	7.3	8.2	8.4	7.9	9.0	8.8			
13	7.4	7.8	8.3	9.8	8.0	8.0	9.2	9.0	8.5			
14	7.7	7.9	7.7	9.3	8.4	7.9	8.7	9.2	9.7			
15	6.8	8.0	8.0	8.4	8.0	9.6	8.6	10.2	9.1			
16	7.4	14.5	8.4	8.0	7.0	10.2	8.7	8.3	9.2			
17	7.3	17.8	8.4	7.3	6.7	9.2	6.7	8.5	9.2			
18	7.3	16.7	7.9	6.7	7.5	9.0	7.7	8.6	8.9			
19	8.0	15.9	8.1	6.8	8.5	8.8	8.0	7.2	8.4			
20	7.6	15.5	8.2	8.4	8.2	8.2	8.9	8.9	8.3			
21	7.9	12.3	7.1	7.7	8.5	9.9	8.4	8.3	10.8			
22	8.1	9.8	7.1	7.5	8.2	10.9	8.2	7.5	9.2			
23	7.7	14.9	7.8	9.5	7.6	9.4	8.3	7.7	8.9			
24	7.5	11.1	7.3	7.0	7.6	9.1	7.4	8.5	8.3			
25	7.6	10.8	8.2	6.7	8.7	9.2	7.0	8.2	9.1			
26	9.7	10.1	8.4	7.3	9.2	8.6	7.1	8.3	8.5			
27	8.2	9.6	8.3	8.3	10.0	7.4	7.9	8.6	8.6			
28	8.0	11.4	8.1	8.2	9.3	7.2	13.6	10.8	9.0			
29	7.9	--	8.5	7.9	8.7	8.3	8.6	8.4	8.7			
30	7.5	--	9.6	7.7	7.8	9.2	8.3	11.1	8.5			
31	7.2	--	8.1	--	7.7	--	7.7	10.5	--			
AVE	7.6	10.6	8.3	7.8	8.1	8.8	8.9	8.9	9.9			

NOTES: 1. Quantities are expressed in $1000m^3$/day

2. Annual average daily flow based upon 9 months measured

= 8.8 x $1000m^3$/day
= 1.94 MIGD

3. Average equivalent flow per Capita = 112.1 gal./day

APPENDIX C

ROGERS CREEK MONITORING PROGRAM

TABLES AND FIGURES

APPENDIX 'C'

ROGERS CREEK MONITORING PROGRAM

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TABLE 1

CREEK WATER QUALITY DATA, JANUARY 21, TO APRIL 2, 1981

Survey No.	Sampling Date	Time	Location	Water Temp. (°C)	Velocity (cm/s)	Depth (cm)	Width (m)	pH	Dissolved Oxygen (mg/l)	Free Ammonia (mg/l)	Nitrogen (as N) Total Kjeldahl (mg/l)	Nitrite (mg/l)	Nitrate (mg/l)	Phosphorus (as P) Total (mg/l)	Dissolved Reactive (mg/l)
1	Jan. 21, 1981	-	100 m u/s WPCP	-1.5	45	23	-	7.6	15.9	1.31	1.68	0.041	3.06	.038	.021
	Jan. 21, 1981	-	50 m d/s WPCP	0.3	45-50	20-25	4.6	7.4	13.1	1.23	1.76	0.320	5.7	.105	.067
	Jan. 21, 1981	-	Hwy. 7 & Rogers Cr	-.05	35-40	18	4.5-6.0	7.3	15.9	1.1	1.89	0.400	5.8	.148	.081
1	Jan. 29, 1981	9:45 a.m.	100 m u/s WPCP	-2	40-50	20-25	7.5-9.6	8.0	15.2	1.13	1.55	.068	2.99	.054	.023
	Jan. 29, 1981	9:55 a.m.	WPCP	4	45	25	-	7.8	12.4	0.136	2.55	.079	18	.660	.360
	Jan. 29, 1981	10:15 a.m.	50 m d/s WPCP	-0.5	35-40	30	9.1	8.1	16.8	0.5	1.65	.320	8.93	.138	.135
	Jan. 29, 1981	12:15 p.m.	Hwy. 7 & Rogers Cr	-0.5	60-70	18	6.1	8.2	15.8	0.74	1.62	.215	7.03	.170	.135
1	Feb. 5, 1981	10:00 a.m.	100 m u/s WPCP	-0.5	20-25	45-60	-	7.65	14.4	1.19	1.60	.036	3.24	.037	.020
	Feb. 5, 1981	10:00 a.m.	WPCP	4.5	20-25	40	-	8.46	13.8	.086	2.10	.030	22	.555	.280
	Feb. 5, 1981	10:00 a.m.	50 m d/s WPCP	1	50-55	20	-	8.7	14.2	.73	1.68	.033	11.8	.220	.155
	Feb. 5, 1981	12:20 p.m.	Hwy. 7 & Rogers Cr	-.5	-	75-90	-	7.45	12.8	.82	1.60	.043	7.66	.148	.086
1	Feb. 12, 1981	9:45 a.m.	100 m u/s WPCP	-0.5	80-85	56	-	7.4	13.6	1.14	2.05	.039	2.36	.128	.067
	Feb. 12, 1981	9:55 a.m.	WPCP	4.0	25	41	-	7.8	13.2	.274	1.78	.088	12	.432	.270
	Feb. 12, 1981	10:15 a.m.	50 m d/s WPCP	-0.5	65	30	-	7.6	13.8	1.06	1.92	.045	3.26	.155	.92
	Feb. 12, 1981	12:35 p.m.	Hwy. 7 & Rogers Cr	-0.5	80	35-40	-	7.4	13.2	.96	2.00	.054	3.65	.170	.105
	Mar. 12, 1981	10:15 a.m.	100 m u/s WPCP	1.5	45-50	30	9.1	8.25	12.2	1.12	1.66	.021	2.23	.059	.036
	Mar. 12, 1981	10:25 a.m.	WPCP	9.	30-35	36	7.6	7.6	11.8	12.1	15.4	.096	3.4	.353	.21
	Mar. 12, 1981	10:30 a.m.	50 m d/s WPCP	1.2	60-75	25	9.1	8.35	12.4	2.0	3.04	1.75	0.60	.098	.043
	Mar. 12, 1981	12:45 p.m.	Hwy. 7 & Rogers Cr	2.	60-70	36	6.1	7.2	12.8	1.87	2.70	.365	1.98	.090	.044
2	April 2, 1981	-	100 m u/s WPCP	7.	110-120	41	-	8.87	12.0	.79	1.55	.042	1.55	.04	.010
	April 2, 1981	-	WPCP	11.	15-20	20	-	7.8	9.8	11.	13.8	.350	6.55	.235	.086
	April 2, 1981	-	50 m d.s WPCP	7	80-90	30	-	8.4	12.0	3.05	4.5	.077	2.62	.098	.035
	April 2, 1981	-	Hwy. 7 & Rogers Cr	10.	65-70	46	-	8.2	12.2	1.29	2.3	.059	1.89	.051	.017

Abbreviations: u/s - upstream from
d/s - downstream from

C-2

TABLE 2

CREEK WATER QUALITY DATA, JANUARY 21 TO APRIL 2, 1981

Survey No.	Sampling Date	Time	Location	Conductivity (umhos/cm)	Chloride as Cl (mg/l)	Total Residual Chlorine (ug/l)	Suspended Solids (mg/l)	Dissolved Solids (mg/l)	Carbonaceous BOD_5 (mg/l)	Nitrogen Prohibited BOD_5 (mg/l)	Total Coliforms (Counts/100 ml)	Background Coliforms (Counts/100 ml)	Fecal Coliforms (Counts/100 ml)	Fecal Strep (Counts/100 ml)
1	Jan. 21, 1981	-	100 m u/s WPCP	1020	118	0	6	-	0.4	-	C870	300	A50	110
	Jan. 21, 1981	-	50 m d/s WPCP	1080	133	0	4	-	0.6	-	2,100	4,800	A60	120
	Jan. 21, 1981	-	Hwy. 7 & Rogers Cr	1120	142	0	5	-	0.8	-	2,400	6,300	A20	A90
1	Jan. 29, 1981	9:45 am	100 m u/s WPCP	980	119	0	8	593	0.4	-	C420	2,900	28	104
	Jan. 29, 1981	9:55 am	WPCP	1600	254	200	4	994	2.0	-	360	940	L10	A30
	Jan. 29, 1981	10:15 am	50 m d/s WPCP	1060	137	8	9	744	0.8	-	1,100	9,000	20	104
	Jan. 29, 1981	12:15 pm	Hwy. 7 & Roger Cr	1100	148	6	11	687	0.6	-	340	1,220	4	148
1	Feb. 5, 1981	10:00 am	100 m u/s WPCP	1040	123	0	6	638	-	-	C420	2,800	24	20
	Feb. 5, 1981	10:00 am	WPCP	1650	262	296	5	1070	-	2.6	430	770	12	58
	Feb. 5, 1981	10:00 am	50 m d/s WPCP	1280	187	118	6	843	-	1.2	380	2,100	8	48
	Feb 5, 1981	12:20 pm	Hwy. 7 & Rogers Cr	1180	157	20	12	751	-	0.8	1,700	9,200	20	32
1	Feb. 12, 1981	9:48 am	100 m u/s WPCP	920	150	0	17	541	-	2.3	C5,100	33,000	256	600
	Feb. 12, 1981	9:55 am	WPCP	1180	206	131	44	737	-	2.2	C10,300	60,000	68	G600
	Feb. 12, 1981	10:15 am	50 m d/s WPCP	940	158	0	17	567	-	2.3	C14,700	26,000	328	G600
	Feb 12, 1981	12:35 pm	Hwy. 7 & Rogers Cr	960	165	0	15	583	-	2.5	C3,900	32,000	296	540
	Mar. 12, 1981	10:15 am	100 m u/s WPCP	820	76	0	8	491	-	0.8	180	2,500	4	8
	Mar. 12, 1981	10:25 am	WPCP	1600	125	234	7	910	-	2.4	1,900	4,100	36	176
	Mar. 12, 1981	10:30 am	50 m d/s WPCP	880	104	7	10	522	-	1.1	C210	3,200	16	L4
	Mar. 12, 1981	12:45 pm	Hwy. 7 & Rogers Cr	900	105	0	2	535	-	0.7	C380	5,500	16	20
2	April 2, 1981	-	100 m u/s WPCP	700	-	0	-	-	-	-	C880	2,800	24	40
	April 2, 1981	-	WPCP	1600	-	472 (240 Mono 184 Di)	-	-	-	-	A40	1,180	8	24
	April 2, 1981	-	50 m d/s WPCP	920	-	54 (0 Mono 46 Di)	-	-	-	-	720	3,800	4	32
	April 2, 1981	-	Hwy. 7 & Rogers Cr	760	-	0	-	-	-	-	C2,040	6,800	60	L4

Abbreviations:
C - background too numerous to count (best estimate given)
A - approximate value, too much dilution
G - actual concentration greater than reported figure
L - actual concentration less than reported
u/s - upstream from
d/s - downstream from

TABLE 3

CREEK WATER QUALITY DATA, APRIL 9 TO JULY 30, 1981

Survey No.	Sampling Date	Time	Location	Water Temp. (°C)	Velocity (cm/s)	Depth (cm)	Width (m)	pH	Dissolved Oxygen (mg/l)	Nitrogen (as N) Free Ammonia (mg/l)	Nitrogen (as N) Total Kjeldahl (mg/l)	Nitrogen (as N) Nitrite (mg/l)	Nitrogen (as N) Nitrate (mg/l)	Phosphorus (as P) Total (mg/l)	Phosphorus (as P) Dissolved Reactive (mg/l)
2	April 9, 1981	-	100 m u/s WPCP	11	110	36	-	8.05	10.0	.056	.54	.160	3.0	.020	.003
	April 9, 1981	-	WPCP	12	20	20	-	8.15	10.2	5.05	7.02	.600	13.4	.135	.020
	April 9, 1981	-	50 m d/s WPCP	11	80	30	-	8.56	12.2	1.37	2.30	E	E	.053	.021
	April 9, 1981	-	Hwy. 7 & Rogers Cr	13	60	30	-	8.8	12.8	.47	1.08	.43	3.67	.039	.009
2	April 14, 1981	-	100 m u/s WPCP	8.5	100	46	-	8.75	12.8	.026	.80	.044	1.91	.098	.030
	April 14, 1981	-	WPCP	11.	20	30	-	7.95	12.2	2.2	7.73	5.2	7.6	.156	.042
	April 14, 1981	-	50 m d/s WPCP	9.	120	30	-	8.36	12.8	.312	2.98	.312	6.59	.115	.032
	April 14, 1981	-	Hwy. 7 & Rogers Cr	9	80	30	-	8.4	12.0	.014	1.16	.010	3.0	.101	.024
2	April 23, 1981	-	100 m u/s WPCP	6	110	-	-	8.45	14.0	.028	.42	.009	2.29	.022	.004
	April 23, 1981	-	WPCP	11	20	-	-	7.35	12.2	.16	7.78	1.68	15.	.161	.022
	April 23, 1981	-	50 m d/s WPCP	8	65	-	-	7.85	13.0	1.17	2.99	.025	7.38	.072	.018
	April 23, 1981	-	Hwy. 7 & Rogers Cr	6	90	-	-	8.5	12.6	.186	1.15	.004	3.8	.040	.011
3	July 23, 1981	10:10 a.m.	100 m u/s WPCP	16	90-95	23	8.2	8.4	11.2	.034	.42	.015	2.03	.019	.007
	July 23, 1981	10:25 a.m.	WPCP	17.5	40	25	6.1	7.7	9.2	.100	1.75	.038	21.5	1.25	.975
	July 23, 1981	10:40 a.m.	50 m d/s WPCP	17.5	70	20	6.1	8.35	12.0	.044	.75	.017	6.49	.288	.238
	July 23, 1981	12:00 noon	Hwy. 7 & Rogers Cr	19.	50	20	7.3	8.2	10.6	.044	.69	.019	7.28	.256	.250
3	July 30, 1981	10:10 a.m.	100 m u/s WPCP	16	100	25	-	8.25	9.5	.004	.46	.027	<.005	.044	.021
	July 30, 1981	10:20 a.m.	WPCP	18	30	25	-	7.7	8.6	.162	1.18	.035	1.77	.285	.035
	July 30, 1981	10:35 a.m.	50 m d/s WPCP	17.5	60	15	-	7.9	9.1	.050	.78	.083	6.18	.122	.082
	July 30, 1981	12:10 p.m.	Hwy. 7 & Rogers Cr	19	55	20	-	8.1	9.7	.004	.72	.093	4.67	.096	.093

Abbreviations: E - sample exhausted
u/s - upstream from
d/s - downstream from

C-4

TABLE 4

CREEK WATER QUALITY DATA, APRIL 9 TO JULY 30, 1981

Survey No.	Sampling Date	Time	Location	Conductivity (umhos/cm)	Chloride asCl (mg/l)	Total Residual Chlorine (ug/l)	Suspended Solids (mg/l)	Dissolved Solids (mg/l)	Carbonaceous BOD_5 (mg/l)	Nitrogen Prohibited BOD_5 (mg/l)	Total Coliforms (Counts/100 ml)	Background Coliforms (Counts/100 ml)	Fecal Coliforms (Counts/100 ml)	Fecal Strep (Counts/100 ml)
2	April 9, 1981	-	100 m u/s WPCP	840	101	0	3	527	-	0.8	C410	4,300	24	40
	April 9, 1981	-	WPCP	1650	295	384	2	1043	-	2.5	C290	3,300	8	24
	April 9, 1981	-	50 m d/s WPCP	1080	209	130	4	654	-	1.5	C350	3,700	4	32
	April 9, 1981	-	Hwy. 7 & Rogers Cr	920	126	0	2	555	-	0.9	A5,000	190,000	60	L4
2	April 4, 1981	-	100 m u/s WPCP	720	84	0	57	415	-	1.4	C5,300	48,000	370	680
	April 4, 1981	-	WPCP	1300	219	464	4	756	-	2.8	C580	3,900	A20	1,070
	April 4, 1981	-	50 m d/s WPCP	900	129	61	37	539	-	2.0	C3,200	31,000	180	680
	April 4, 1981	-	Hwy. 7 & Rogers Cr	740	89	0	47	445	-	1.3	15,000	68,000	420	490
2	April 23, 1981	-	100 m u/s WPCP	170	-	0	4	467	-	0.6	C980	6,000	120	296
	April 23, 1981	-	WPCP	160	-	62 (10 Mono 52 Di)	5	972	-	2.8	C1,240	4,800	16	408
	April 23, 1981	-	50 m d/s WPCP	1100	-	17 (0 Mono 17 Di)	34	638	-	2.2	C1,060	6,600	76	344
	April 23, 1981	-	Hwy. 7 & Rogers Cr	880	-	0	<1	519	-	1.1	C8,300	39,000	170	210
3	July 23, 1981	10:10 am	100 m u/s WPCP	920	126	0	1	563	-	0.8	C1,100	42,000	200	450
	July 23, 1981	10:25 am	WPCP	1500	233	129	9	987	-	2.6	A40	2,400	L10	L10
	July 23, 1981	10:40 am	50 m d/s WPCP	1050	165	11	2	651	-	2.2	C360	21,000	A40	240
	July 23, 1981	12:00 noon	Hwy. 7 & Rogers Cr	1080	156	3	4	648	-	1.1	C13,700	60,000	6,900	14,200
3	July 30, 1981	10:10 am	100 m u/s WPCP	860	105	0	4	480	-	0.5	C850	4,100	1,500	1,000
	July 30, 1981	10:20 am	WPCP	1340	232	195	2	788	-	1.0	C440	5,400	A20	A10
	July 30, 1981	10:35 am	50 m d/s WPCP	1020	144	30	4	577	-	0.8	4,300	23,000	750	450
	July 30, 1981	12:10 pm	Hwy. 7 & Rogers Cr	960	128	0	5	582	-	0.3	10,000	90,000	1,600	A450

Abbreviations: C - background too numerous to count (best estimate given)
A - approximate value, too much dilution
L - actual concentration less than reported
u/s - upstream from
d/s - downstream from

TABLE 5

CREEK WATER QUALITY DATA, AUGUST 5 TO OCTOBER 21, 1981

Survey No.	Sampling Date	Time	Location	Water Temp. (°C)	Velocity (cm/s)	Depth (cm)	Width (m)	pH	Dissolved Oxygen (mg/l)	Nitrogen (as N) Free Ammonia (mg/l)	Total Kjeldahl (mg/l)	Nitrite (mg/l)	Nitrate (mg/l)	Phosphorus (as P) Total (mg/l)	Dissolved Reactive (mg/l)
3	Aug. 5, 1981	10:37 a.m.	100 m u/s MPCP	19	50	30	6.1	8.1	8.5	.006	.82	.002	1.8	.155	.038
	Aug. 5, 1981	11:02 a.m.	MPCP	19.5	.4	15	-	8.3	8.6	.008	.92	.002	11.	.272	.175
	Aug. 5, 1981	11:14 a.m.	50 m d/s MPCP	19.5	34	36	5.5	7.98	8.65	.006	.36	.001	4.55	.075	.080
	Aug. 5, 1981	12:04 p.m.	Hwy. 7 & Rogers Cr	20	30	18	6.1	6.9	8.7	.006	.35	.001	3.15	.064	.065
3	Aug. 13, 1981	-	100 m u/s MPCP	19	80	-	-	8.3	10.8	.004	.62	.002	1.35	.070	.031
	Aug. 13, 1981	-	MPCP	20	20	-	-	7.6	8.2	.008	1.18	NA	NA	.819	.80
	Aug. 13, 1981	-	50 m d/s MPCP	19	70	-	-	8.05	9.9	.026	.90	NA	NA	.362	NA
	Aug. 13, 1981	-	Hwy. 7 & Rogers Cr	21	80	-	-	8.4	9.8	.008	.75	.003	3.8	.172	.15
4	Sept. 30, 1981	10:10 a.m.	100 m u/s MPCP	8	85	-	-	8.15	12.8	.016	.40	.006	2.35	.024	.002
	Sept. 30, 1981	10:10 a.m.	MPCP	15	-	-	-	7.75	8.7	.014	2.18	1.72	12.0	.230	.077
	Sept. 30, 1981	10:20 a.m.	50 m d/s MPCP	10.5	70	-	-	8.0	11.0	.030	1.22	.083	10.0	.145	.055
	Sept. 30, 1981	12:10 p.m.	Hwy. 7 & Rogers Cr	10	35	-	-	7.7	11.8	<.002	.58	.021	4.98	.063	.032
4	Oct. 7, 1981	-	100 m u/s MPCP	9	120	-	-	8.1	11.7	.006	.55	.015	.140	.050	.024
	Oct. 7, 1981	-	MPCP	15	-	-	-	7.8	9.6	.014	.82	.053	15.0	.175	.125
	Oct. 7, 1981	-	50 m d/s MPCP	11	85	-	-	8.0	11.6	.016	.82	.012	2.79	.100	.069
	Oct. 7, 1981	-	Hwy. 7 & Rogers Cr	9	70	-	-	8.5	11.6	.006	.70	.003	2.05	.065	.035
4	Oct. 13, 1981	-	100 m u/s MPCP	5	80	-	-	6.9	13.6	.008	.38	.010	1.99	.014	.002
	Oct. 13, 1981	-	MPCP	15	-	-	-	6.5	9.6	.004	.69	.001	15.8	.181	.135
	Oct. 13, 1981	-	50 m d/s MPCP	6	75	-	-	6.8	13.2	.002	.40	.025	2.1	.017	.012
	Oct. 13, 1981	-	Hwy. 7 & Rogers Cr	7	35	-	-	7.3	13.0	<.002	.51	.001	4.7	.058	.019
4	Oct. 21, 1981	-	100 m u/s MPCP	7	115	45	-	7.9	12.0	.032	.49	.011	1.39	.036	.013
	Oct. 21, 1981	-	MPCP	10	50	50	-	6.8	11.5	.008	1.12	.400	10.6	.145	.045
	Oct. 21, 1981	-	50 m d/s MPCP	9	100	35	-	7.3	11.8	.006	.74	.120	6.07	.081	.034
	Oct. 21, 1981	-	Hwy. 7 & Rogers Cr	8	75	35	-	7.9	12.0	.010	.60	.064	2.89	.043	.024

Abbreviations: NA - not available
u/s - upstream from
d/s - downstream from

C-6

TABLE 6

CREEK WATER QUALITY DATA, AUGUST 5 TO OCTOBER 21, 1981

Survey No.	Sampling Date	Time	Location	Conductivity (umhos) cm)	Chloride asCl (mg/l)	Total Residual Chlorine (ug/l)	Suspended Solids (mg/l)	Dissolved Solids (mg/l)	Carbonaceous BOD_5 (mg/l)	Nitrogen Prohibited BOD_5 (mg/l)	Total Coliforms (Counts/ 100 ml)	Background Coliforms (Counts/ 100 ml)	Fecal Coliforms (Counts/ 100 ml)	Fecal Strep (Counts/ 100 m)
3	Aug. 5, 1981	10:37 am	100 m u/s WPCP	760	84	-	66.3	475	-	1.2	45,000	63,000	35,000	A7,000
	Aug. 5, 1981	11:02 am	WPCP	1060	188	-	51.9	726	-	0.8	680	830	A70	A10
	Aug. 5, 1981	11:14 am	50 m d/s WPCP	1060	131	-	52.8	578	-	2.0	31,000	83,000	A9,000	A1,000
	Aug. 5, 1981	12:04 pm	Hwy. 7& Rogers Cr	840	106	-	71.4	518	-	1.8	55,000	132,000	1,460	480
3	Aug. 13, 1981	-	100 m u/s WPCP	800	70	0	10.5	492	-	0.7	C3,500	36,000	A400	A200
	Aug. 13, 1981	-	WPCP	1600	238	288	11.8	987	-	2.2	A30	120	L10	A10
	Aug. 13, 1981	-	50 m d/s WPCP	1140	137	22	8.0	685	-	1.4	L10	110	A10	A20
	Aug. 13, 1981	-	Hwy. 7 & Rogers Cr	820	97	0	11.4	573	-	0.6	C3,000	34,000	320	250
4	Sept. 30, 1981	10:00 am	100 m u/s WPCP	910	126	0	3.8	572	-	0.4	1,300	9,900	A80	A90
	Sept. 30, 1981	10:10 am	WPCP	1000	258	252	4.5	969	-	3.0	160	1,700	A20	A40
	Sept. 30, 1981	10:20 am	50 m d/s WPCP	1260	197	94	3.7	829	-	0.8	C380	3,800	A50	A30
	Sept. 30, 1981	12:10 pm	Hwy. 7 & Rogers Cr	1100	137	0	2.5	670	-	0.4	31,000	43,000	330	A80
4	Oct. 7, 1981	-	100 m u/s WPCP	730	70	0	11.7	452	-	0.8	2,600	17,500	580	750
	Oct. 7, 1981	-	WPCP	1220	196	137	2.1	788	-	0.5	160	2,250	A10	L10
	Oct. 7, 1981	-	50 m d/s WPCP	950	131	10	7.6	612	-	0.4	1,200	14,500	280	340
	Oct. 7, 1981	-	Hwy. 7 & Rogers Cr	780	85	0	8.4	492	-	0.5	11,000	100,000	920	620
4	Oct. 13, 1981	-	100 m u/s WPCP	820	97	0	1.6	501	-	0.4	C6,700	23,000	530	A70
	Oct. 13, 1981	-	WPCP	400	244	39	1.8	260	-	10.5	1,500	7,300	290	100
	Oct. 13, 1981	-	50 m d/s WPCP	840	99.5	0	2.5	530	-	0.3	C580	3,400	310	A90
	Oct. 13, 1981	-	Hwy. 7 & Rogers Cr	920	118	0	1.8	593	-	0.4	5,500	21,000	390	110
4	Oct. 21, 1981	-	100 m u/s WPCP	720	64	0	4	440	-	0.5	1,300	14,000	370	110
	Oct. 21, 1981	-	WPCP	1140	60	63	3	753	-	0.6	1,800	7,100	640	120
	Oct. 21, 1981	-	50 m d/s WPCP	960	118	8	10	592	-	0.6	1,600	11,600	940	A90
	Oct. 21, 1981	-	Hwy. 7 & Rogers Cr	780	79.5	0	6	486	-	0.5	C5,000	25,000	740	200

Abbreviations: C - background too numerous to count (best estimate given)
A - approximate value, too much dilution
L - actual concentration less than reported
u/s - upstream from
d/s - downstream from

TABLE 7

FLOWS 1981

M^3/s

DATE	WPCP	ROGERS CREEK
January 29	0.093	0.600
February 5	0.086	0.265
February 12	0.094	0.560
July 23	0.095	0.244
July 30	0.100	0.450
August 5	0.167	0.470
August 13	0.112	0.580

FIGURE 1 - WATER QUALITY SAMPLING LOCATIONS

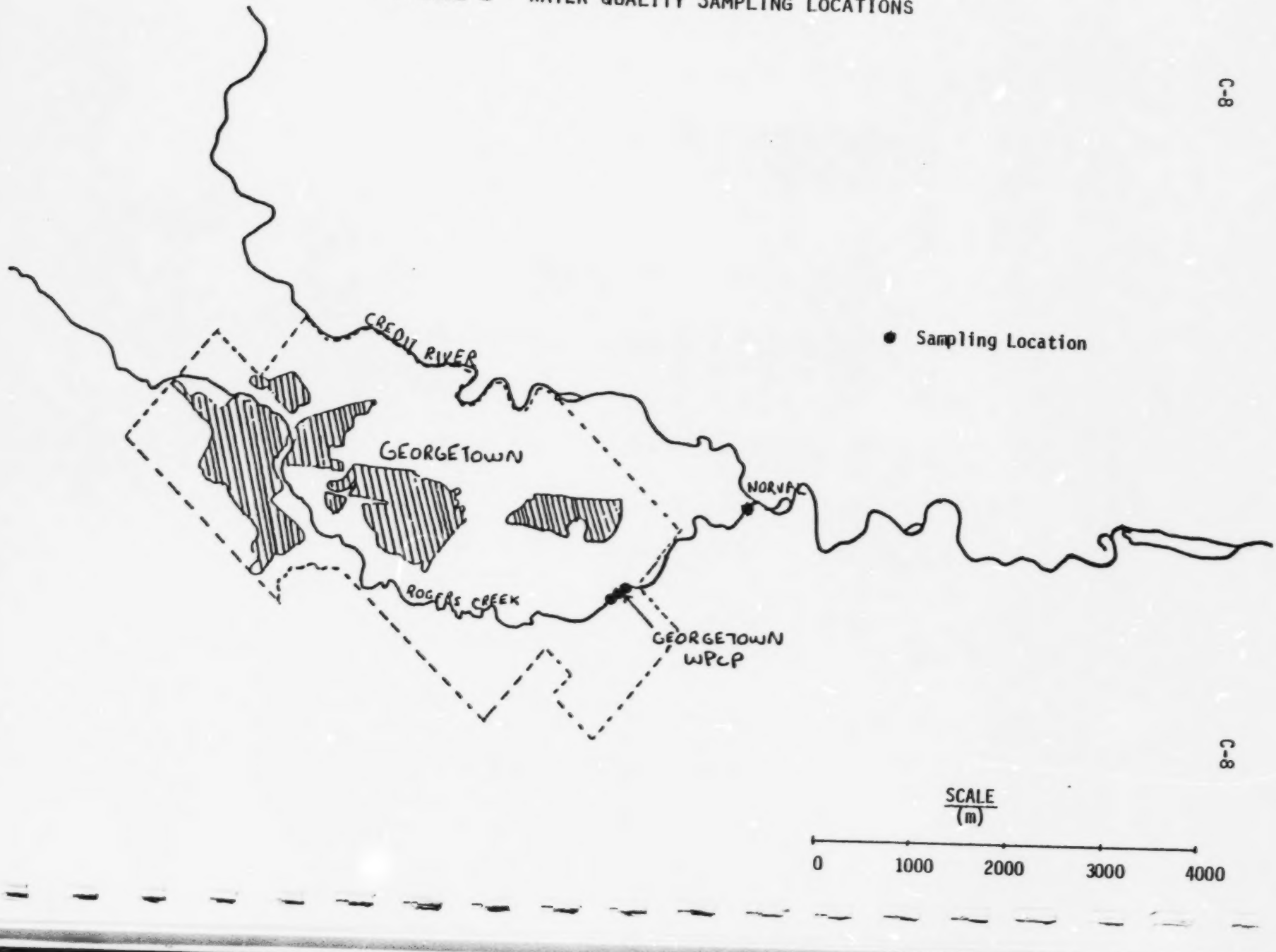


FIGURE 2 - CREEK CROSS-SECTIONS

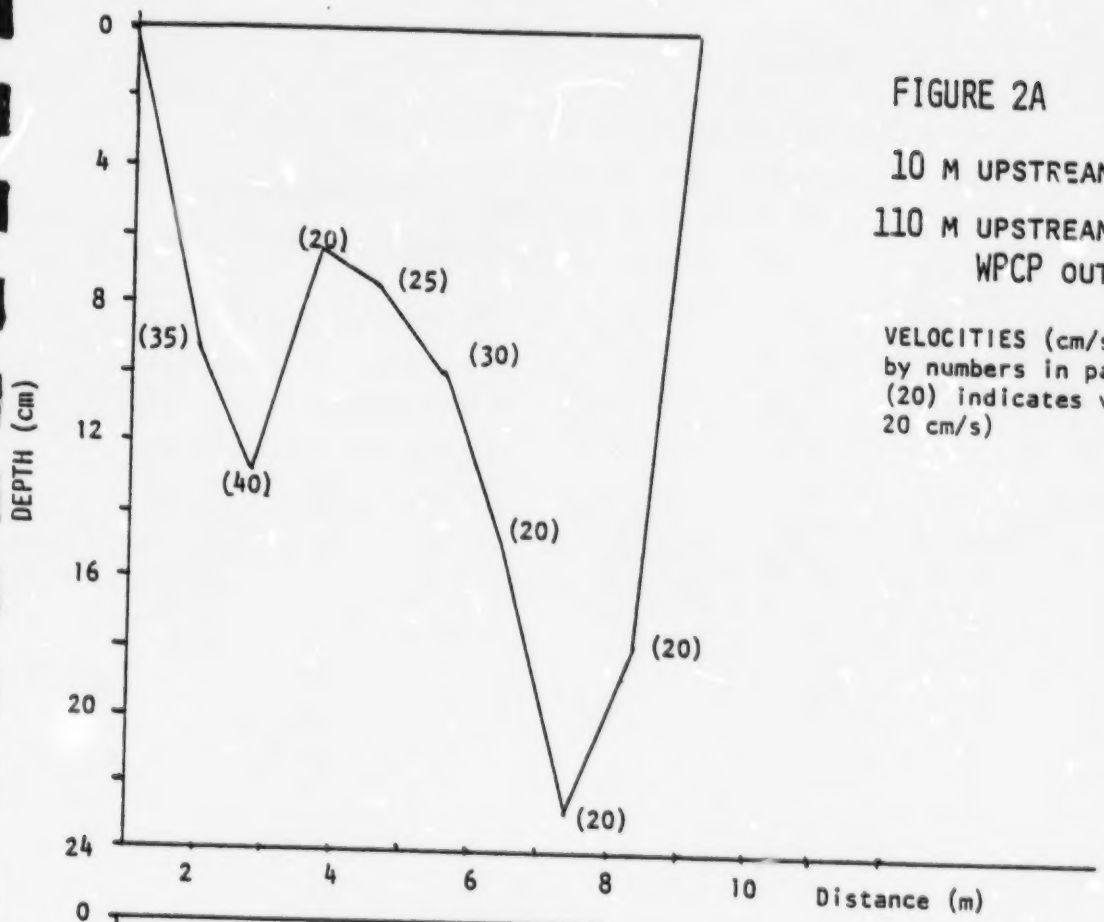


FIGURE 2A

10 M UPSTREAM OF STATION 1

110 M UPSTREAM OF GEORGETOWN WPCP OUTFALL

VELOCITIES (cm/s) are indicated by numbers in parentheses (e.g. (20) indicates velocity of 20 cm/s)

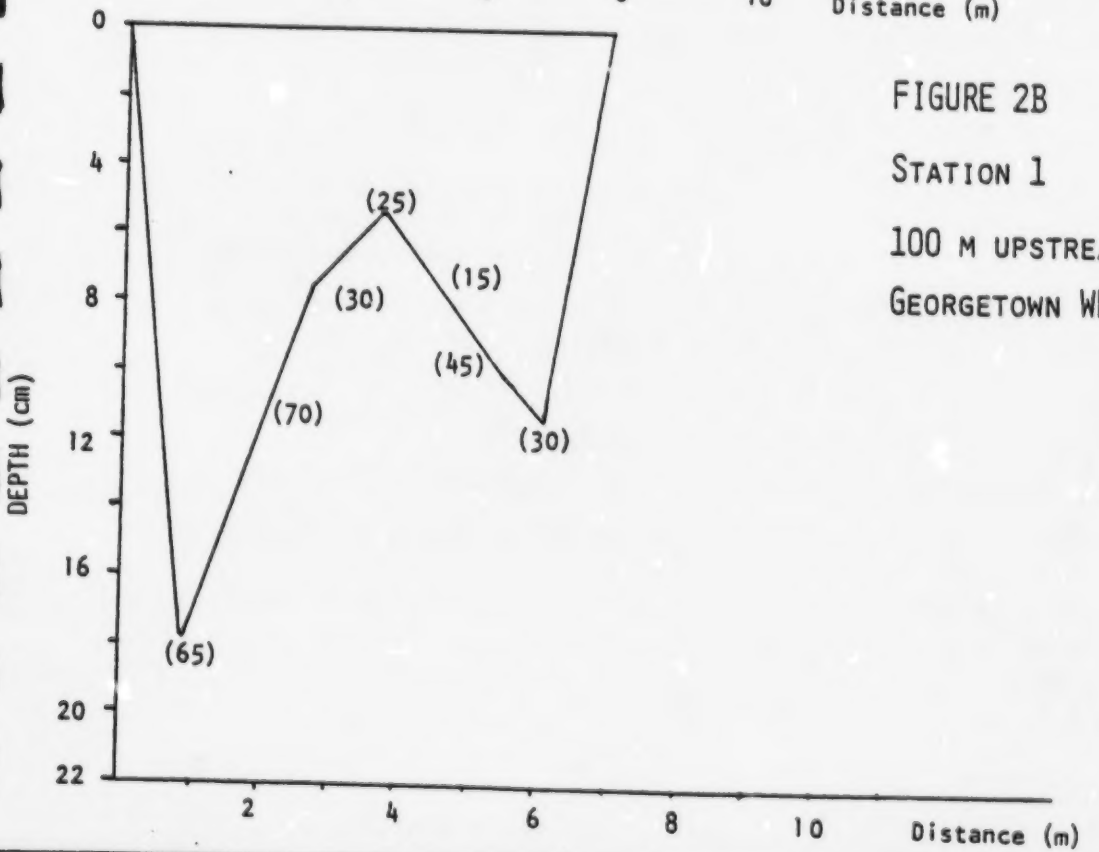


FIGURE 2B

STATION 1

100 M UPSTREAM FROM GEORGETOWN WPCP OUTFALL.

FIGURE 3 - CREEK CROSS-SECTIONS

C-10

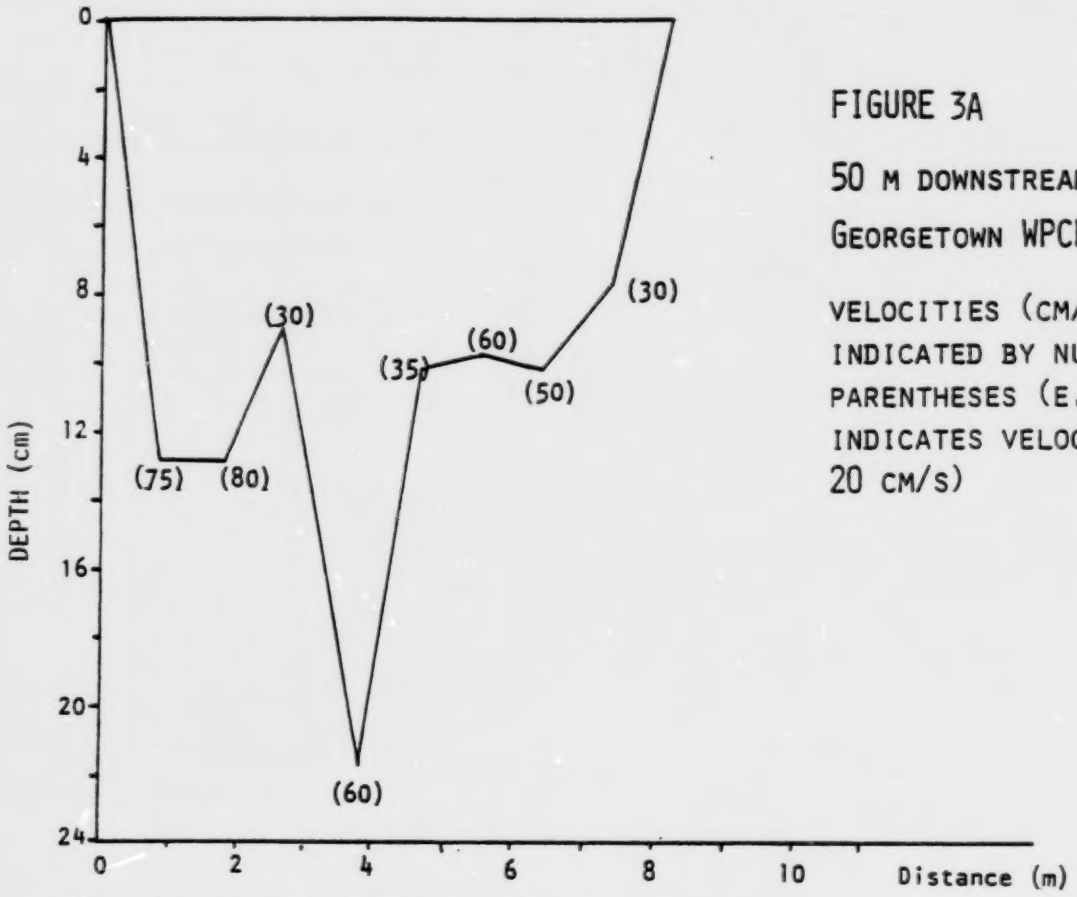


FIGURE 3A

50 M DOWNSTREAM OF GEORGETOWN WPCP OUTFALL

VELOCITIES (CM/S) ARE INDICATED BY NUMBERS IN PARENTHESES (E.G. (20) INDICATES VELOCITY OF 20 CM/S)

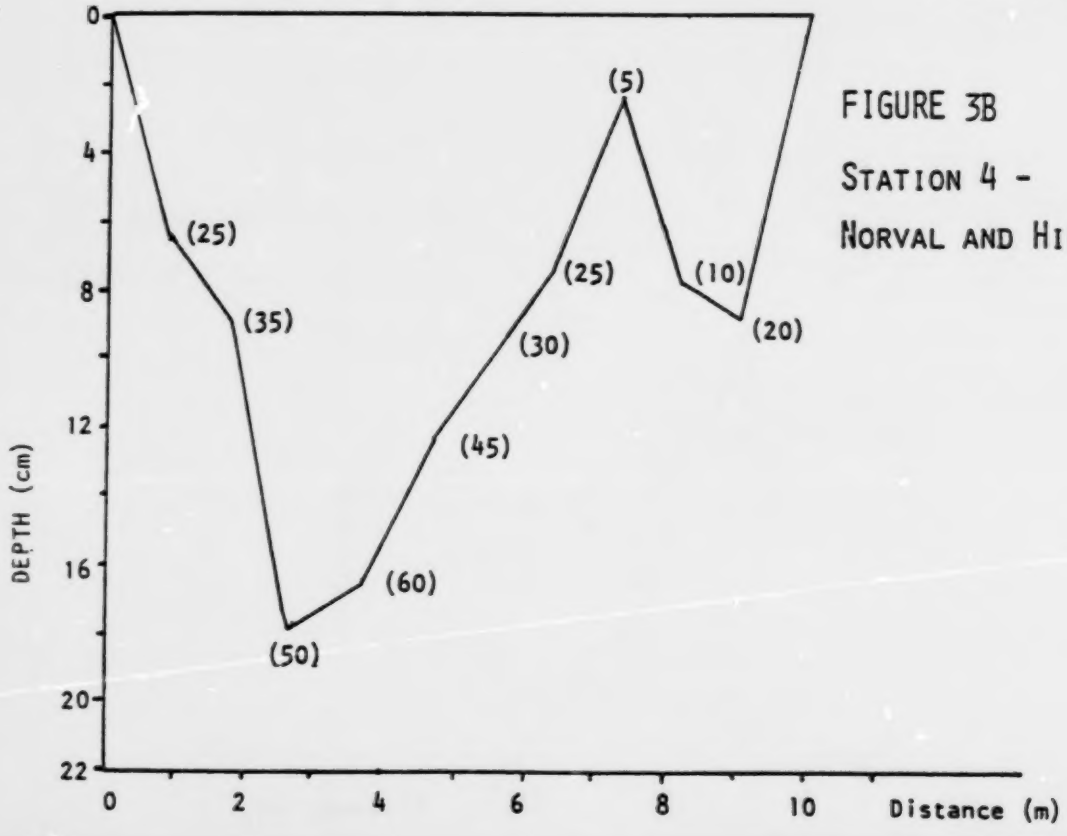
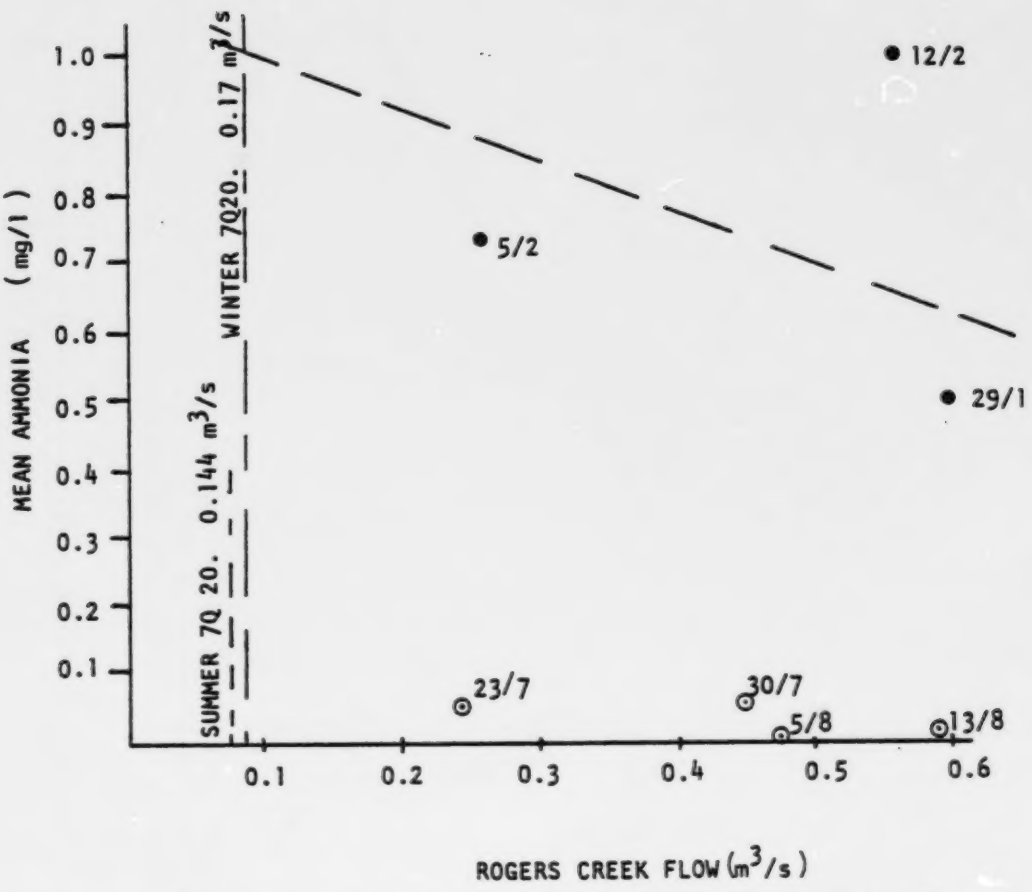


FIGURE 3B

STATION 4 - NORVAL AND HIGHWAY 7

FIGURE 4 - ROGERS CREEK AMMONIA CONCENTRATIONS DOWNSTREAM OF THE EFFLUENT DISCHARGE OF 0.09 AND 0.11 M^3/S IN THE WINTER AND SUMMER, RESPECTIVELY.



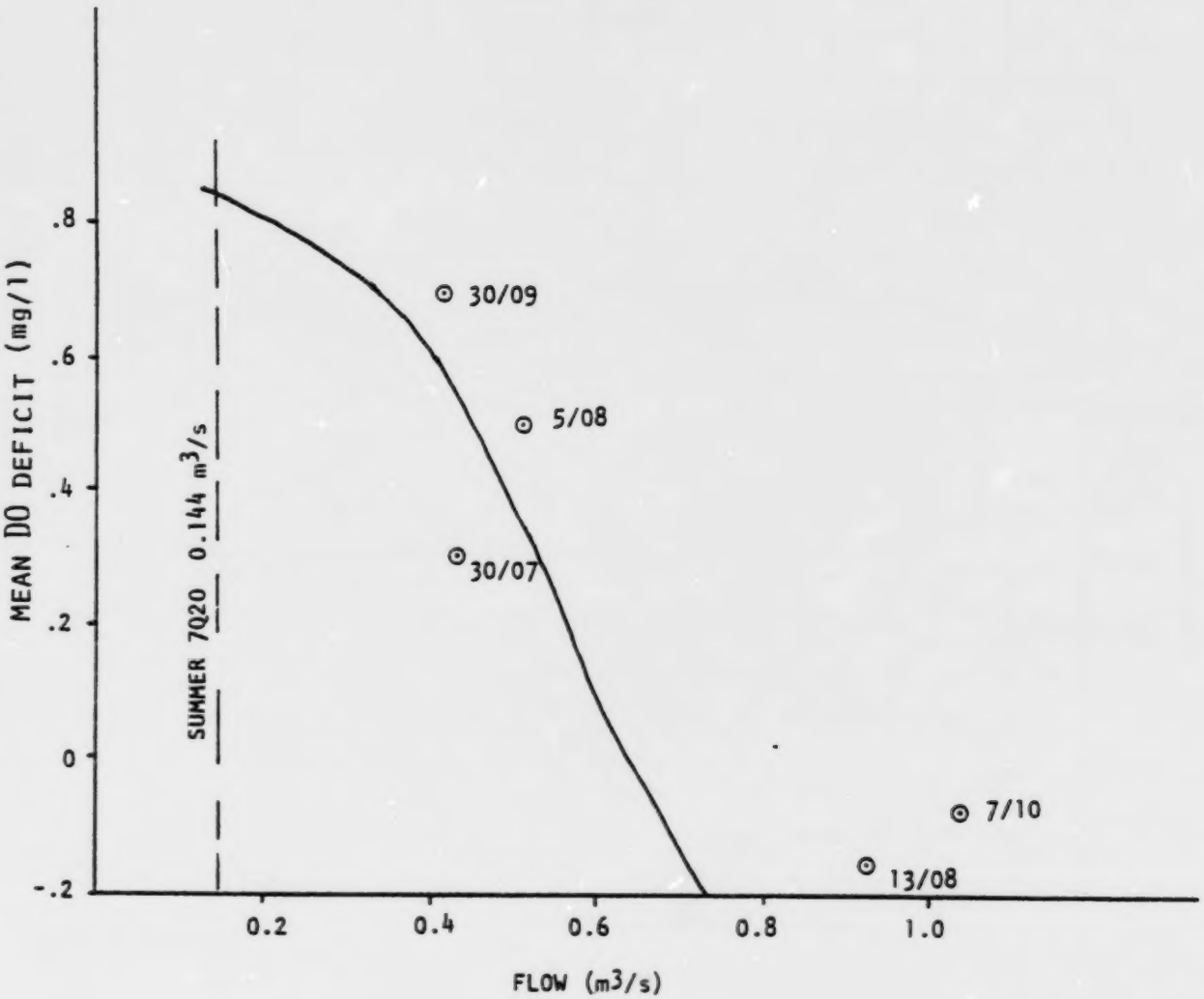
- 1981 Sampling Data (day/month) is noted beside each point.

● Winter data point

⊙ Summer data point

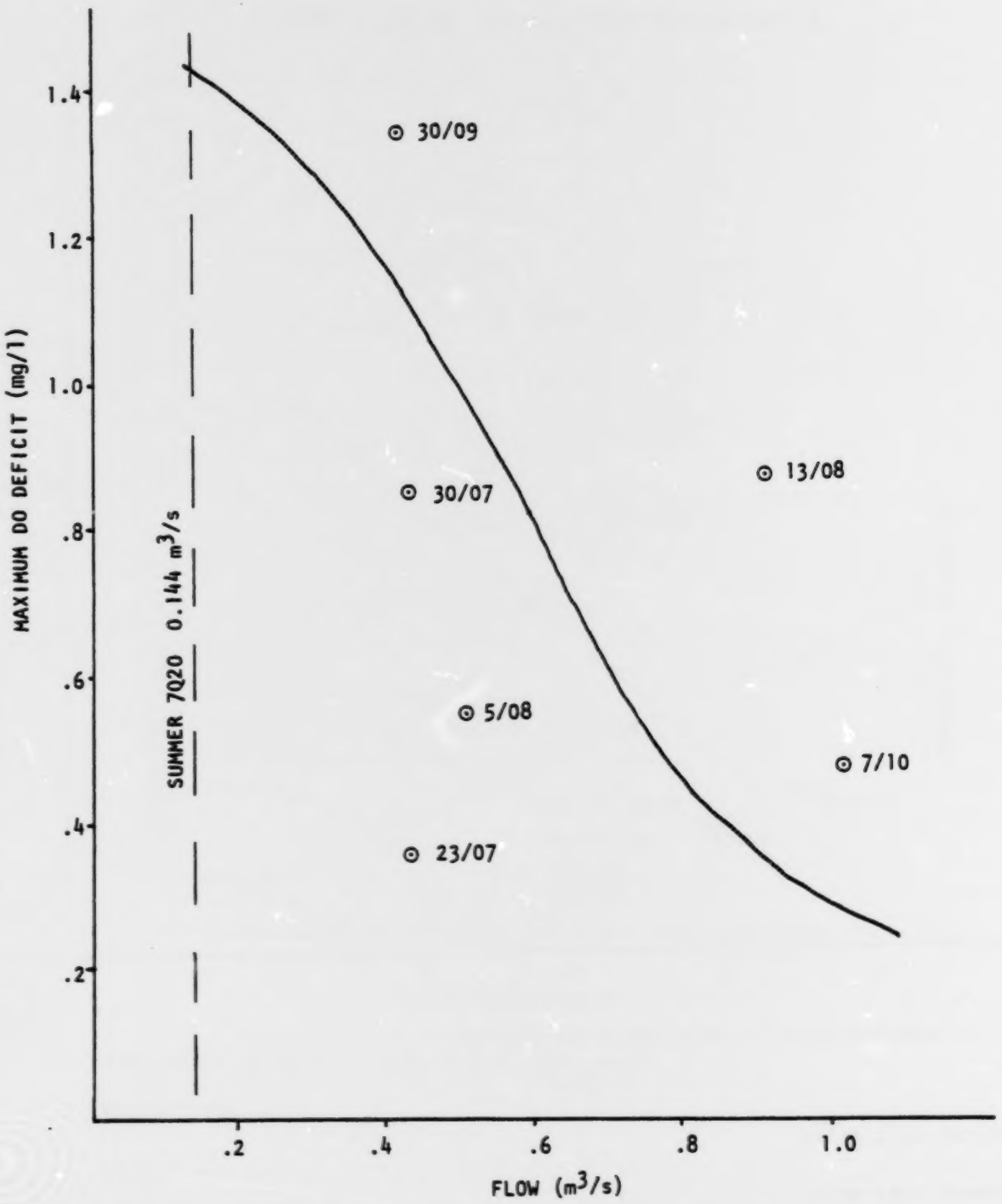
C-12

FIGURE 5 - ROGERS CREEK MEAN DISSOLVED OXYGEN DEFICIT DOWNSTREAM OF THE EFFLUENT DISCHARGE OF 0.09 AND 0.11 m^3/s IN THE WINTER AND SUMMER, RESPECTIVELY.



- 1981 Sampling Date (day/month) is noted beside each point

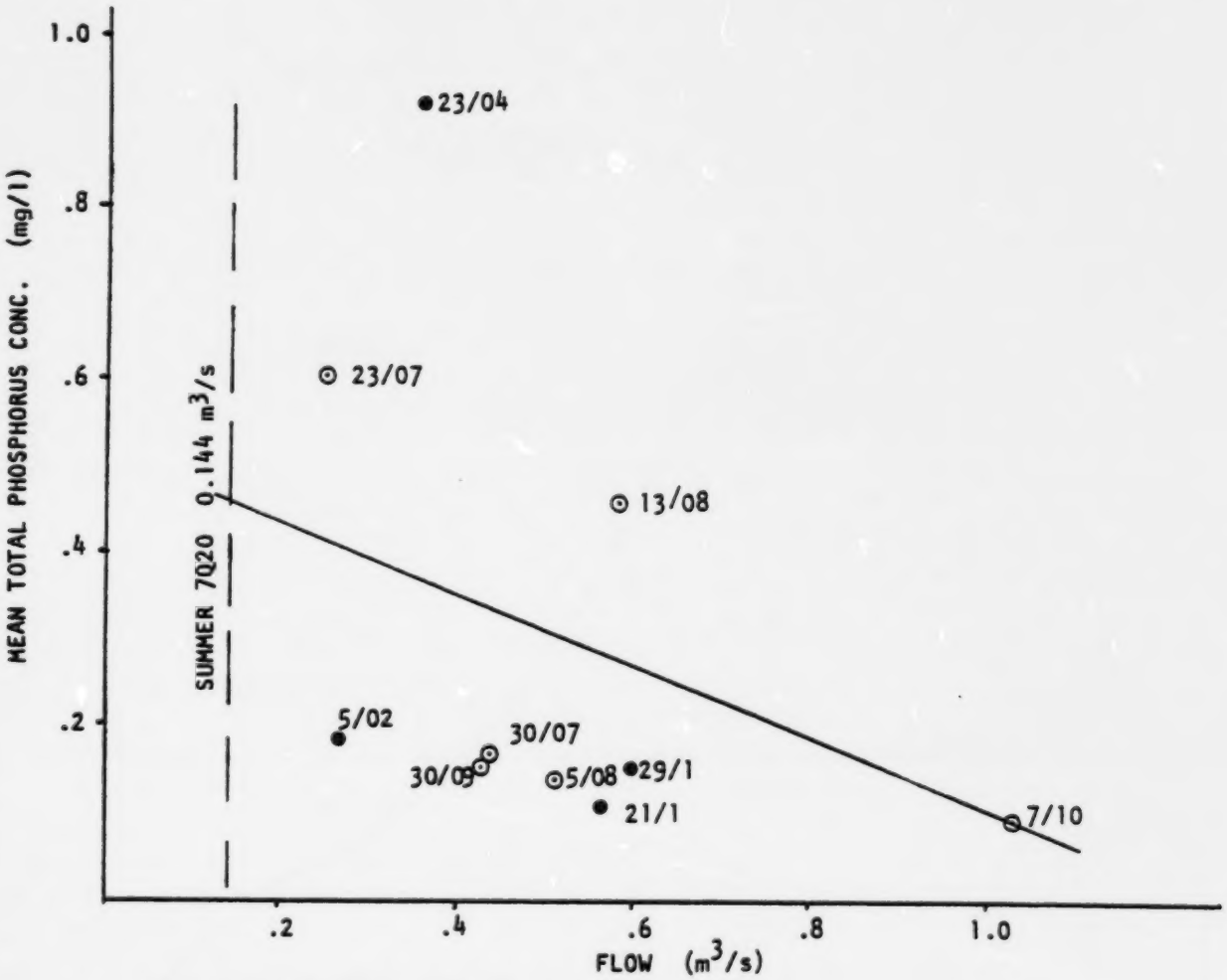
FIGURE 6 - ROGERS CREEK MAXIMUM DISSOLVED OXYGEN DEFICIT DOWNSTREAM OF THE EFFLUENT DISCHARGE OF 0.09 AND 0.11 M^3/S IN THE WINTER AND SUMMER, RESPECTIVELY.



- 1981 Sampling Date (day/month) is shown beside each point.

C-14

FIGURE 7 - ROGERS CREEK MEAN TOTAL PHOSPHORUS DOWNSTREAM OF THE EFFLUENT DISCHARGE OF 0.09 AND 0.11 m^3/s IN THE WINTER AND SUMMER, RESPECTIVELY.

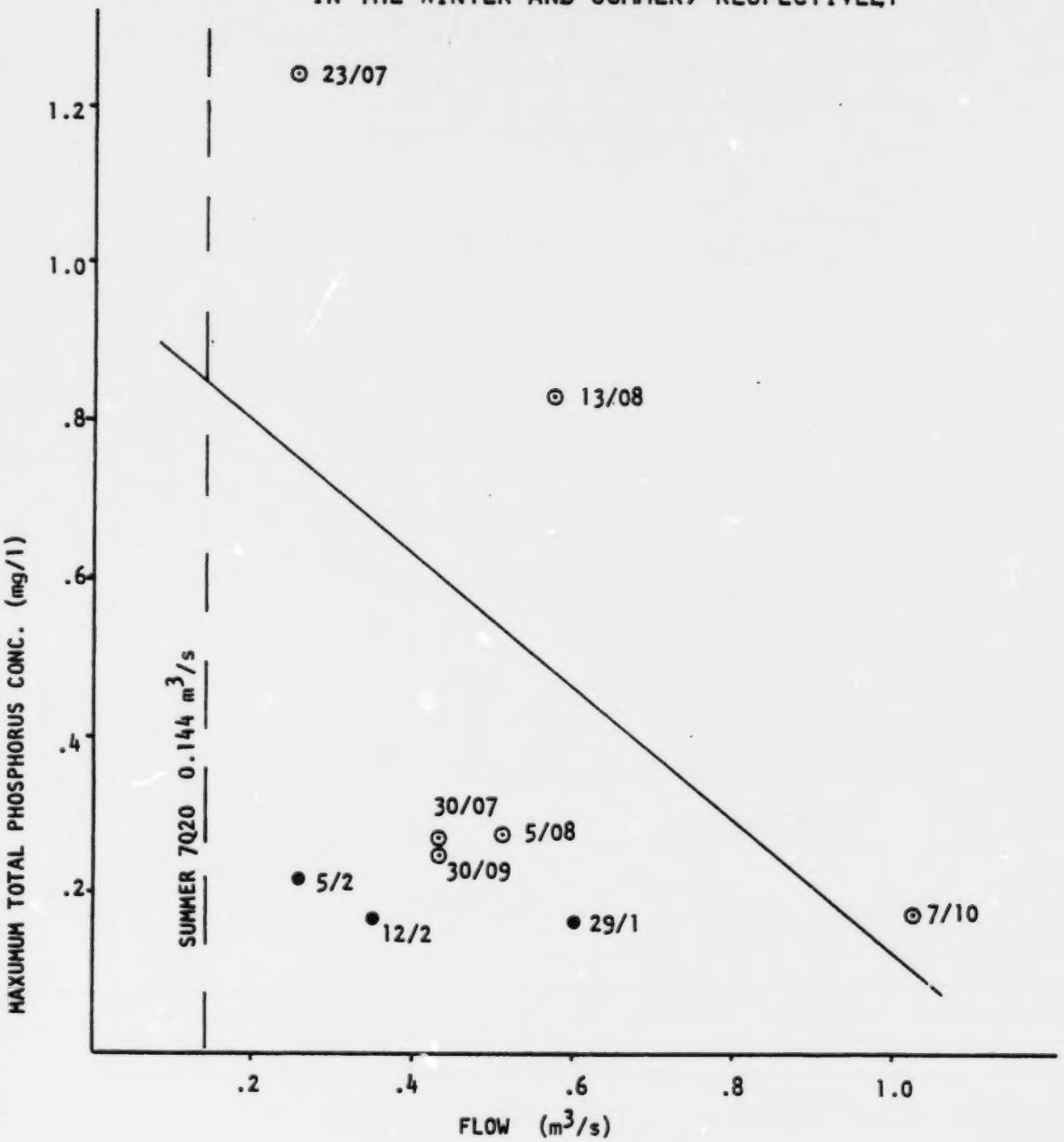


- 1981 Sampling Date is shown beside each point.

● Winter data point

⊙ Summer data point

FIGURE 8 - ROGERS CREEK MAXIMUM TOTAL PHOSPHORUS DOWNSTREAM OF THE EFFLUENT DISCHARGE OF 0.09 AND 0.11 M^3/s IN THE WINTER AND SUMMER, RESPECTIVELY



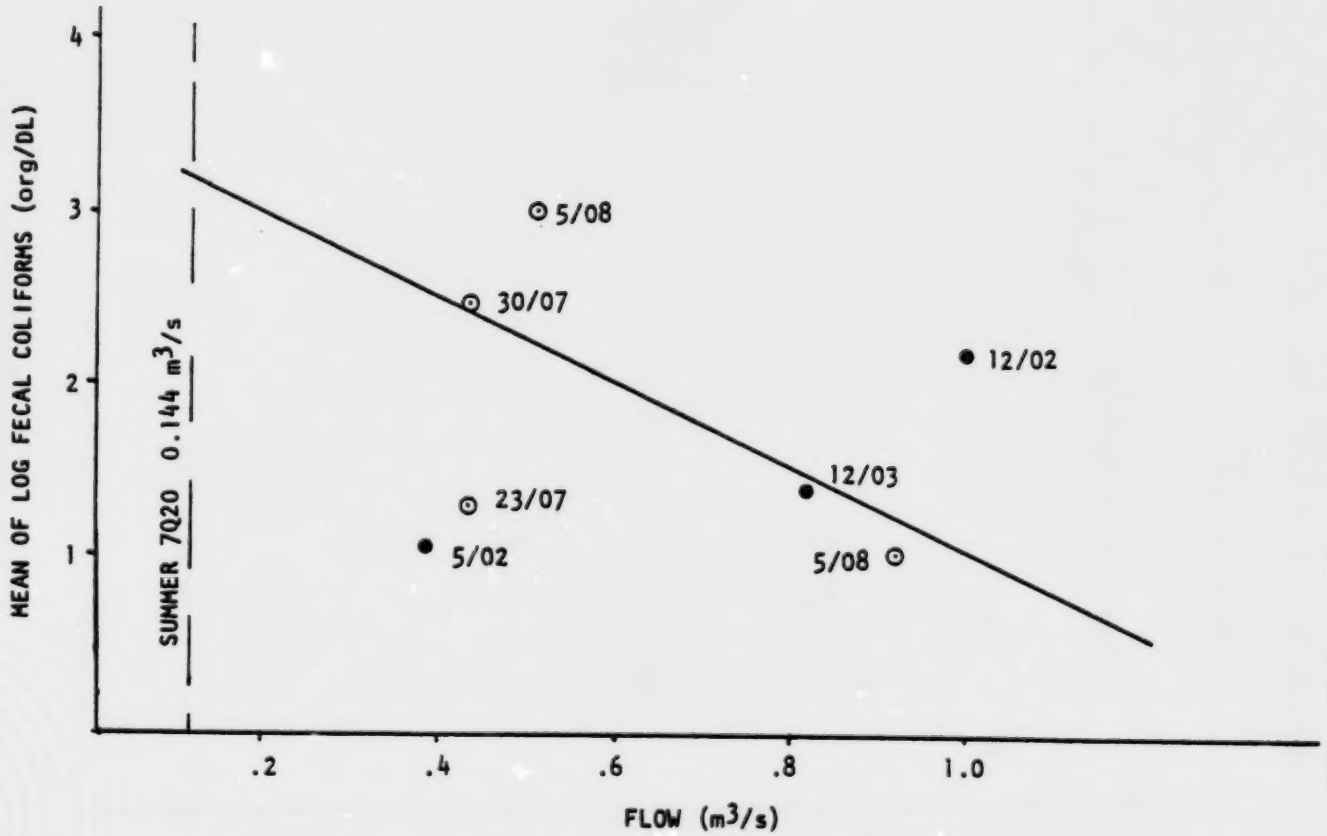
- 1981 Sampling Date is shown beside each point

● Winter data point

⊙ Summer data point

C-16

FIGURE 9 - ROGERS CREEK MEAN LOG FECAL COLIFORMS DOWNSTREAM OF THE EFFLUENT DISCHARGE OF 0.09 AND 0.11 m^3/s IN THE WINTER AND SUMMER, RESPECTIVELY.

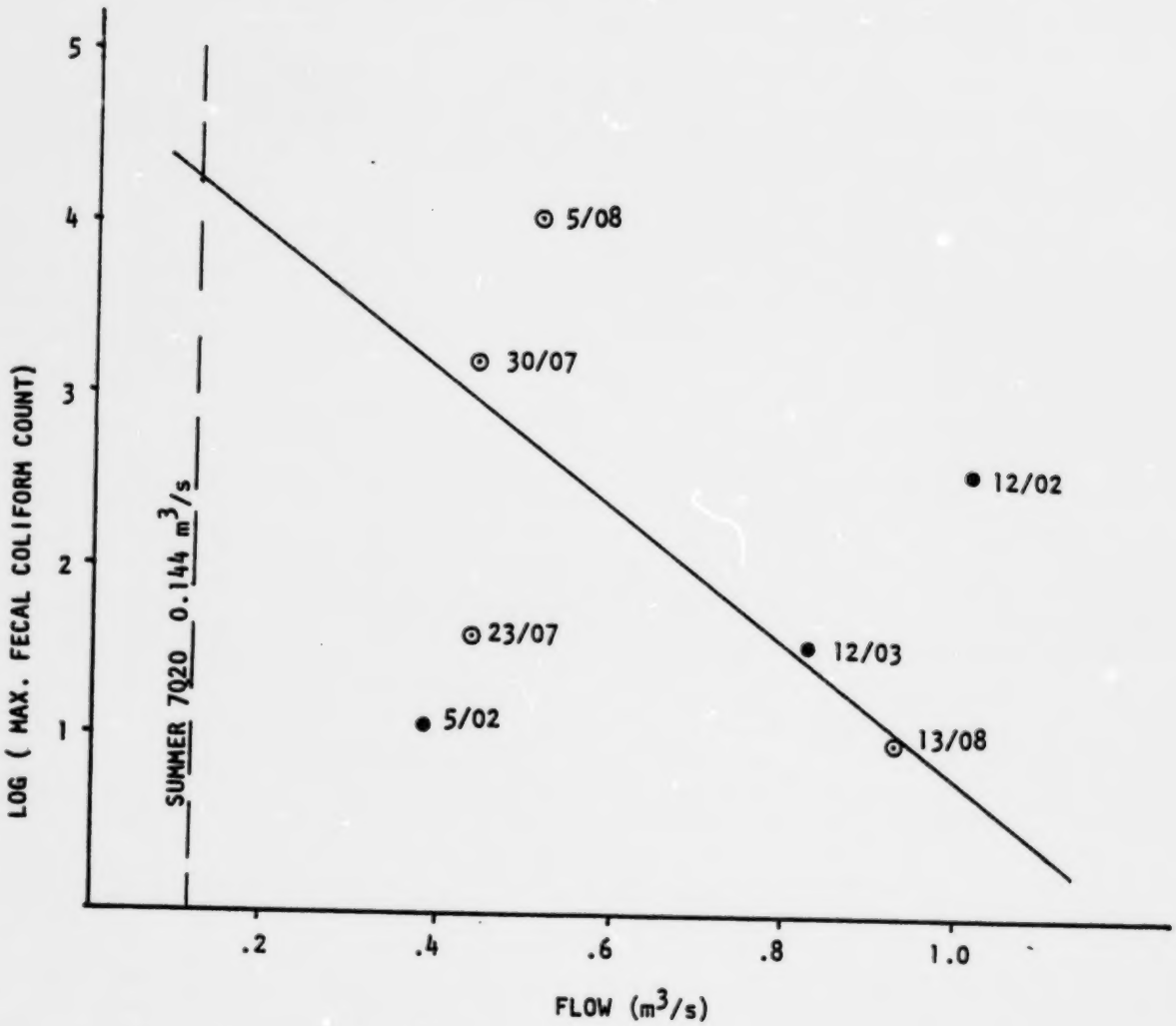


- 1981 Sampling Date (day/month) is shown beside each point.

● Winter data point

⊙ Summer data point

FIGURE 10 - ROGERS CREEK MAXIMUM LOG FECAL COLIFORMS DOWNSTREAM OF THE EFFLUENT DISCHARGE OF 0.09 AND 0.11 M^3/S IN THE WINTER AND SUMMER, RESPECTIVELY.



- 1981 Sampling Date (day/month) is shown beside each point.

● Winter data point

⊙ Summer data point

APPENDIX D

BACKGROUND REPORT
STAGE II
HALTON HILLS (GEORGETOWN)
URBAN AREA REVIEW
NOVEMBER 12, 1981

BACKGROUND REPORT
STAGE II
HALTON HILLS (GEORGETOWN)
URBAN AREA REVIEW
REGIONAL MUNICIPALITY OF HALTON

PN 5824/2
31 July 1981
REVISED
23 September 1981
12 November 1981

MUNICIPAL PLANNING CONSULTANTS
DIVISION OF MPC LIMITED
TOWN PLANNERS PROFESSIONAL ENGINEERS

SECTION 1 INTRODUCTION

(1) BACKGROUND

Prior to this Study, the Ministry of the Environment had established the Georgetown sewage treatment plant capacity at an equivalent population of 30,000 persons. With the recent pace of new development in the Georgetown area, it was suspected that the 30,000 equivalent population had been reached or possibly surpassed. As certain lands within the former Town had been approved by Council for development, terms of reference were prepared for the Urban Area Review Study. The Study was to be conducted in 2 stages:

(a) STAGE I:

Stage I of the Study involves a review and evaluation of the Georgetown sewerage and sewage treatment systems to analyze the factors governing performance and means of improving the systems so as to increase capacity; and

(b) STAGE II:

Stage II of the Study includes the planning and other engineering parts of the project. Its purpose is to determine the equivalent population of existing and committed development and to translate any identified surplus sewage treatment capacity into future development and boundary recommendations.

On January 21, 1981, the Regional Municipality of Halton authorized Municipal Planning Consultants to undertake preliminary work for Stage II. The program for the preliminary work includes aspects of the Planning Study (Stage II) which could be undertaken independent of the outcome of the Engineering Study (Stage I).

(2) STUDY AREA AND SERVICE AREAS

The Study Area includes those lands designated by the Region of Halton Official Plan as being within the Georgetown Urban Area as well as other adjacent lands which are the subject of Regional Official Plan referrals or deferrals. Precise Study Area boundaries are shown on the maps appended to this Report.

To provide more detail, some of the data presented in this Report are set out by service area. These service areas were defined in a previous engineering study of Georgetown[1] and are shown on the attached Appendix A.

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[1] SOURCE: Georgetown Sanitary Sewerage Study, Proctor & Redfern Ltd., 1977.

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(3) OBJECTIVES

The objectives for the Stage II preliminary work are to:

(a) assess, verify and update the data base necessary to the project as required;

(b) ascertain the nature of physical constraints on development for vacant lands in the Study Area;

(c) assess the suitability of current Study Area boundaries with respect to existing and committed developments;

(d) determine the equivalent population of existing and committed development within the Study Area;

(e) provide appropriate liaison between the Consultant and the Steering Group, involved agencies and other affected parties; and

(f) conduct all aspects of the Study so as to be fully prepared to provide expert evidence at any related hearings of the Ontario Municipal Board.

(4) DATA BASE

(a) COMPILATION AND REVIEW:

Statistical data and other information were compiled for the Study Area regarding such factors as population, household size, existing land use, physical development constraints, environmental issues, property ownership, commercial and industrial land and building inventories, sewage treatment capacity, building activity, and proposed development. A comprehensive review of existing studies, reports and other documents prepared by or for the Region and the Town was conducted so as to determine their relevance to the Urban Area Review. Several of these studies had been undertaken in preparation of the Region of Halton Official Plan, addressing planning and engineering issues in the Georgetown area such as:

(i) population dynamics and growth targets;
(ii) traffic and transportation policies;
(iii) housing;
(iv) environmentally sensitive areas and flood plains;
(v) recreation planning;

(vi) hamlet secondary plans;
(vii) capital budgeting;
(viii) sewage treatment and water supply;
(ix) retail market potential; and
(x) future development capacity.

It was determined that certain aspects of the data base required verification and updating. Field survey data compiled for the project was utilized as required to verify available information.

(b) LAND USE SURVEY:

A land use survey of the Study Area was undertaken in April 1981 and compared with previous (1976) land use information and a review of building permits. The land use survey data aided in checking household counts and locations of undeveloped properties, in compiling a new inventory of land use for that portion of the Study Area lying outside the former Town limits, and in obtaining additional information not available through other means. Existing land use mapping is attached to this Report as Appendix B.

(5) STUDY ASSUMPTIONS

(a) SEWAGE TREATMENT CAPACITY:

It has been determined that, subject to corrective measures now under study in Stage I, the present sewage treatment plant would have sufficient capacity to handle an average daily flow of greater than that for which it is now rated. Ministry of Environment officials have acknowledged that the revised capacity may be obtainable, subject to review of the final findings of Stage I. For the purposes of this Report, the capacity is assumed to be 5 million gallons per day.

(b) FLOW CONTRIBUTION RATES:

From the findings of Stage I to date, an equivalent flow contribution factor of 125 gallons per capita per day (gcd), including an allowance for infiltration, has been established. When dealing with specific land uses, this Report uses the following load factors which were also derived from Stage I work.

residential load:	61 gallons/capita/day
commercial load:	3,560 gallons/acre/day
industrial load:	2,440 gallons/acre/day

SECTION 2 EXISTING DEVELOPMENT

(1) RESIDENTIAL

The assessed population of the former Town of Georgetown is given as 17,300. To check this figure and to provide other important demographic data, this Report has determined an everage household size by service area for each type of dwelling unit (Table 1) and then multiplied those figures by the number of dwelling units of a given type to derive a population for each service area (Table 2). The total population derived in this manner (17,076) is within 2% of the assessed population (17,300). The reason for the discrepancy is likely due to an under-counting of additional units in what appear to be one-family homes.

Those portions of the Study Area outside the former Town limits had a 1979 population of just under 900.

TABLE 1 HOUSEHOLD SIZE BY SERVICE AREA (1979)
GEORGETOWN URBAN AREA
TOWN OF HALTON HILLS[1]

	HOUSEHOLD SIZE (persons per unit)			
SERVICE AREA NO.	1 FAMILY	2 FAMILY	MULTIPLE	APARTMENT
1, 2	3.72	3.48	1.86	2.12
3	3.89	2.32	2.49	
4	3.6	3.94	1.75	2.33
5, 6	3.3			2.45
7, 8, 9	3.29	3.58	2.39	1.45
10	3.58	3.5	2.25	
11, 12, 13				
14, 16, 17	3.14	2.83	2.92	2.09
15, 18	2.92	3.03	2.4	1.88
19	3.51	3.25	2.49	2.01
20	3.28	3.38	3.24	2.22
Rural Portion	3.28	3.38	3.24	2.22
AVERAGE:	3.51	3.25	2.49	2.01

[1] SOURCE: Inventory and Density Characteristics by Seven Structure Types, Regional Information Systems Committee (based on assessment statistics), 1979.

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TABLE 2 DWELLING UNITS AND POPULATION (1979)
GEORGETOWN URBAN AREA
TOWN OF HALTON HILLS

SERVICE AREA NO.	DWELLING UNITS[1] 1 FAMILY	2 FAMILY	MULTIPLE	APARTMENT	POPULATION[2]
1	557	122	29	72	2,704
2	547	-	31	-	2,092
3	485	-	73	-	2,068
4	7	6	4	40	149
5	522	-	-	-	1,722
6	362	-	-	117	1,482
7	149	16	8	-	568
8	44	13	15	-	229
9	324	24	33	-	1,230
10	257	-	-	-	920
11	135	16	14	-	510
12	107	-	3	6	358
13	56	-	-	-	185
14	107	18	17	-	437
15	195	42	24	60	869
16	86	4	4	82	466
17	13	-	86	-	293
18	19	12	7	-	110
19	40	25	-	225	674
20	3	-	-	-	10
URBAN SUBTOTALS:	4,015	298	348	602	17,076
Rural Portion	242	-	29	-	888
TOTALS:	4,257	298	377	602	17,964

[1] SOURCE: Field Survey, Municipal Planning Consultants, April 1981.

[2] NOTE: These figures are calculated by multiplying dwelling units by household size figures from Table 1.

(2) COMMERCIAL DEVELOPMENT

The gross commercial floorspace data shown in Table 3 were compiled from:

- the land use survey;
- the 1975 and 1976 Georgetown Commercial Floorspace Inventories;
- the review of 1975-1981 building permits; and
- a telephone survey and field study for data unavailable from any other source.

Although there has been a noticeable degree of commercial relocation within the Study Area since 1975, this Report assumes that 1975-1976 commercial floorspace is still existing. This assumption is based on the high degree of consistency established through a comparison of 1976 and 1981 land use data.

(3) INDUSTRIAL DEVELOPMENT

Table 3 also sets out gross industrial floorspace data for the Study Area as compiled from a number of sources, including:

- the land use survey;
- the 1975 Town of Halton Hills Industrial Survey;
- the January 1976 Industrial List of Georgetown, issued by the Georgetown Chamber of Commerce;
- the November 1977 Realty Research Group Ltd. Study: Industrial Space Growth Analysis Georgetown, Ontario;
- the November 1977 Real Search Corporation Study: Land Value Survey of Industrial Lands in the Former Town of Georgetown;
- a review of building permits for the 1975-1981 period; and
- a comprehensive telephone survey of existing industries.

Response to the industrial survey was excellent. Information from respondents provided a good check on data available from alternative sources.

TABLE 3 COMMERCIAL AND INDUSTRIAL FLOORSPACE (1981)
GEORGETOWN URBAN AREA
TOWN OF HALTON HILLS[1]

SERVICE AREA NO.	GROSS FLOOR SPACE (x1,000 sq. ft.) COMMERCIAL	INDUSTRIAL
1	136	103
2	184	-
3	11	-
4	146	1,320
5	-	-
6	61	57
7	-	-
8	17	-
9	128	3
10	13	-
11	2	2
12	-	-
13	-	-
14	-	218
15	13	3
16	-	280
17	-	36
18	7	-
19	41	13
20	-	-
URBAN SUBTOTALS:	759	2,035
Rural Portion	7	-
TOTALS:	766	2,035

[1] SOURCE: Sources of information for Table 3 are described in Sections 2(2) and 2(3) of this Report.

(4) HAMLETS

Municipal water is currently supplied from the Georgetown system to all or part of the 3 hamlets of Glen Williams, Norval and Stewarttown. In 1977, the Town of Halton Hills Council adopted a resolution for the allocation of sewage treatment capacity to the 3 hamlets.[1] The servicing of the hamlet populations seemed imminent, as problems with septic system performance were known to exist in these areas.

The combined 1979 assessed population of the 3 hamlets was 2,075 persons (Glen Williams - 1,325; Norval - 430; Stewarttown - 320). While these areas are primarily residential communities, some sewage treatment capacity allowance may be appropriate not only to recognize existing commercial and industrial land uses, but also to recognize the potential future development of currently vacant sites. Utilizing the flow contribution factor of 125 gcd, 259,375 gallons of treatment capacity would be required in reserve for the hamlets, representing 5.2% of the total capacity of a 5 million gallon/day treatment plant.

(5) STUDENT POPULATION

It is common practice to include an allowance of 15 gcd for sewage flows generated by student populations. In this Report, it is assumed that sewage flows generated by students attending school in the Study Area are accounted for in the overall flow contribution factor.

(6) CAPACITY ALLOCATED

Table 4 summarizes the populations and sewage treatment capacity allocations for existing development in the Study Area as well as the reserve for the 3 hamlets. The total allocation represents 50.7% of the potential treatment plant capacity as determined in Stage I.

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[1] NOTE: Even though parts of these hamlets are outside the Study Area, each of the hamlets is considered in its entirety.

TABLE 4 EXISTING DEVELOPMENT
GEORGETOWN URBAN AREA
TOWN OF HALTON HILLS

LOCATION	POPULATION	SEWAGE TREATMENT ALLOCATION (gal/day)
Urban Portion of Study Area	17,300	2,162,500
Rural Portion of Study Area	888	111,000
SUBTOTALS:	18,188	2,273,500
Hamlets	2,075	259,375
TOTALS:	20,263	2,532,875

SECTION 3 COMMITTED DEVELOPMENT

(1) DEFINITION

Additional sewage treatment capacity is required in reserve for the servicing of developments to which the Municipality is formally committed. For the purposes of this Study, a "commitment" has been defined to include:

- projects under construction;
- proposals covered by draft-approved plans; and
- proposals for which special exemptions to zoning have been granted by Council.

Committed developments are shown on Appendix D.

Vacant parcels zoned for commercial or industrial use but for which there are no site-specific proposals which have been approved by the Town have not been considered as committed development even though such parcels may be part of a registered plan of subdivision. Development of these parcels is not considered to be committed since non-residential development is usually subject to site plan controls and other approvals. These parcels are, however, addressed later in this Report in determining additional development capability.

(2) RESIDENTIAL DEVELOPMENT

As of July 1981, there were 16 sites within the former Town boundaries committed to some form of residential development. It is also expected that the International Bible Students Association (IBSA) complex, to the north of Moore Park, will provide single apartment accommodation for 200 persons. A summary of the projects by dwelling unit type and service area is set out in Table 5.

Utilizing the household size figures from Table 1, the population increase expected from the projects in Table 5 is 2,427 persons. However, for calculating required treatment capacity, a population of 2,227 will be used since the 200 persons at the IBSA complex will not use the Georgetown sanitary sewerage system. At a flow contribution rate of 61 gcd for residential land uses, this population will generate a demand for 135,847 gallons of sewage treatment capacity or 2.96% of the total capacity of a 5 million gallon/day treatment plant.

TABLE 5 COMMITTED RESIDENTIAL DEVELOPMENT
GEORGETOWN URBAN AREA
TOWN OF HALTON HILLS[1]

SERVICE AREA NO.	DWELLING UNITS 1 FAMILY	2 FAMILY	MULTIPLE	APARTMENT	POPULATION[2]
1			60		217
2	127				472
4				158	368
5	10				33
6		22			71
9	14				46
13	6				19
	46				145
	80				251
14	2				6
16			46		134
	6	42	27		217
		8			23
19				42	85
	12				42
	28				98
URBAN SUBTOTALS:	345	72	133	200	2,227
Rural Portion				200	200[3]
TOTALS:	345	72	133	400	2,427

[1] SOURCE: Development Listing, Town of Halton Hills Planning Dept., 1981.

[2] NOTE: These population figures are estimates calculated from household size figures in Table 1.

[3] NOTE: The International Bible Students Association residential facilities consist of single apartment dwelling units for 200 persons.

(3) COMMERCIAL DEVELOPMENT

There are 6 vacant sites within the former Town limits currently zoned for commercial construction or expansion. One site of 0.82 acres is under construction, plans are approved for development of one 0.5 acre site, 2 sites are zoned for commercial expansions, and the remaining 2 are zoned for commercial use although currently vacant. The first 4 of the above are considered to be committed at present and are summarized in Table 6. Utilizing the commercial flow contribution factor of 3,560 gallons/acre/day, the committed 12.22 acres should generate a sewage treatment demand of 43,503 gallons/day. This demand represents 0.87% of a total potential capacity of 5 million gallons/day.

The International Bible Students Association complex contains an additional 132,100 square feet of commercial floor space in administrative office, printing and other service activities. However, this facility will not require any sewage treatment capacity at the Georgetown plant since it will have its own sewage treatment facilities.

TABLE 6 COMMITTED COMMERCIAL DEVELOPMENT
GEORGETOWN URBAN AREA
TOWN OF HALTON HILLS

SERVICE AREA NO.	LOT SIZE (acres)	SEWAGE TREATMENT DEMAND (gal/day)[1]	MAXIMUM BUILDING AREA (sq. ft.)[2]
1	0.5	1,780	5,500
2	8.3	29,548	90,400
4	2.6	9,256	28,300
16	0.82	2,920	8,945
URBAN SUBTOTALS:	12.22	43,503	133,145
Rural Portion	n/a [3]	nil [4]	132,100 [5]
TOTALS:	12.22	43,503	265,245

1 NOTE: Lot size (acres) x 3,560 gallons/acre/day.
2 NOTE: Calculated from provisions of Zoning By-law No. 57-91.
3 NOTE: Not applicable because site is mixed use.
4 NOTE: Nil because site will have own treatment works.
5 NOTE: Actual building area proposed.

(4) INDUSTRIAL DEVELOPMENT

Current commitments for the development of industrial land are set out in Table 7, together with the anticipated sewage treatment demand. Utilizing the industrial flow contribution rate of 2,440 gallons/acre/day derived from Stage I, the development commitments on 38.86 industrial acres will require reserving 94,818 gallons/day of treatment capacity. An organic loading equivalent of an additional 375,000 gallons/day is expected from the proposed expansion of the Neilson operations, thus requiring a total reserve of 469,818 gallons/day for committed industrial sites. This demand represents 9.4% of a total potential capacity of 5 million gallons/day.

Although the foregoing figures are not easily convertible into precise industrial floor space, it seems likely that development of the above industrial sites could add approximately 480,000 square feet of industrial floor space to the existing inventory. Details of floor space calculations are set out in Appendix C.

TABLE 7 COMMITTED INDUSTRIAL DEVELOPMENT
GEORGETOWN URBAN AREA
TOWN OF HALTON HILLS

SERVICE AREA NO.	LOT SIZE (acres)	SEWAGE TREATMENT DEMAND (gal/day)[1]	ANTICIPATED FLOOR SPACE (sq. ft.)[2]
1	6.93	16,909	45,281
	2.83	6,905	49,310
4	2.53	6,173	44,083
	14.45	35,258	94,416
	3.52	8,589	61,332
	1.51	3,684	26,310
17	7.09	17,300	46,326
SUBTOTALS:	38.86	94,818	367,058
Neilson	43.42	375,000 [3]	115,000 [3]
TOTALS:	82.28	469,818	482,058

[1] NOTE: Lot size (acres) x 2,440 gallons/acre/day.
[2] NOTE: Calculated from assumptions described in Appendix C.
[3] SOURCE: Consultations, Gore & Storrie Limited, 1981

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(5) CAPACITY AND POPULATION SURPLUS

The equivalent population of existing and committed development within the Study Area is 25,456 persons, based on the overall flow contribution factor of 125 gcd. An equivalent population surplus of 14,544 remains, given a total sewage treatment plant capacity of 5 million gallons/day. The sewage treatment allocations for existing and committed development and their equivalent populations are set out in Table 8.

TABLE 8 EXISTING AND COMMITTED DEVELOPMENT
GEORGETOWN URBAN AREA
TOWN OF HALTON HILLS

FLOW SOURCE	SEWAGE TREATMENT ALLOCATION (gal/day)	EQUIVALENT POPULATION[1]
Committed Residential	135,847	
Committed Commercial	43,503	
Committed Industrial	469,818	
COMMITTED SUBTOTAL:	649,168	5,193
Existing Development (Study Area)	2,273,500	18,188
Hamlet Reserve	259,375	2,075
TOTALS:	3,182,043	25,456
POTENTIALS: [2]	5,000,000	40,000
SURPLUS:	1,817,957	14,544

[1] NOTE: Based on flow contribution factor of 125 gcd.

[2] NOTE: These figures are from preliminary findings of Stage I.

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SECTION 4 ADDITIONAL DEVELOPMENT CAPABILITY

There are several sites within the former Town boundaries which have not yet been committed to some form of urban development. They are shown on Appendix D. The ultimate development of these sites will impact on sewage treatment capacity and may influence the assessment of boundary suitability since they represent future development potential. Some of these additional sites are readily developable while others are constrained by physical factors, ownership or lack of access.[1] For purposes of assessing this development potential, these sites are inventoried below. Developable sites are classified on the basis of Official Plan designations.

(1) RESIDENTIAL SITES

There are 8 sites which could potentially be developed for residential use. The estimated number of units, anticipated populations and expected sewage treatment requirements are set out in Table 9.

TABLE 9 ADDITIONAL RESIDENTIAL SITES
GEORGETOWN URBAN AREA
TOWN OF HALTON HILLS

SERVICE AREA	DWELLING UNITS 1 FAMILY[2]	DWELLING UNITS TOWNHOUSE[3]	POPULATION[4]	SEWAGE TREATMENT ALLOCATION (gal/day)
5	10		31	
6		127	316	
7	32		105	
9	4		13	
11	35		110	
	4		13	
13	11		35	
19	35		123	
TOTALS:	131	127	746	45,506

1 NOTE: Mapping of physical development constraints in the Study Area is attached as Appendix E.

2 NOTE: Derived from lot area (acres) x 6 units/acre.

3 NOTE: Derived from lot area (acres) x 15 units/acre.

4 NOTE: These household figures are estimates calculated from household size figures in Table 1.

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(2) COMMERCIAL SITES

There are 2 vacant sites, with a combined area of 0.9 acres, currently zoned for commercial use within the former Town. At a flow contribution rate of 3,560 gallons/acre/day, 3,204 gallons/day of treatment capacity would be required to service these sites.

(3) INDUSTRIAL SITES

At present, 33.73 acres of vacant land are zoned for industrial use. At a flow contribution rate of 2,440 gallons/acre/day, 82,301 gallons/day of treatment capacity would be required to service these sites.

(4) CONSTRAINED SITES

Within the former Town, there are 5 undeveloped sites which have one or more constraints on their future development.

(a) MEADOWGLEN MUSHROOM PROPERTY:

This 41-acre site is designated for low density residential development in the Official Plan. After allowing for roads and park dedication, this site could accommodate 210 single-detached units (at 6 units/net acre) or approximately 740 persons at 3.51 persons/household.[1] This population would require 45,140 gallons/day of sewage treatment capacity.

(b) DOMINION SEED PROPERTY:

The 60.86-acre Dominion Seed property is a designated Special Study Area and was considered in the recent Guelph Street Corridor Study. The future land use recommended for the property in that study was medium density residential. At 15 units/acre, this site could accommodate approximately 750 dwelling units, or a population of 2,630 persons at 3.51 person/household. This population would demand 160,430 gallons/day of sewage treatment capacity.

---

[1] NOTE: This figure represents the 1979 overall average household size for Georgetown as determined from Provincial assessment data.

(c) McLAREN FARM:

This 17-acre site is designated for industrial use in the Official Plan. At a rate of 2,440 gallons/acre/day, development of this farm property would generate a demand for 41,480 gallons/day of sewage treatment capacity.

(d) MAPLE AVENUE EAST:

This 2.5-acre farm property is also designated for industrial use in the Official Plan. The sewage treatment requirement in this case would be 6,100 gallons/day.

(e) CARRUTHERS ROAD:

This 9.55-acre site is currently landlocked by the Canadian National Railway holdings and the former Town's western boundary, along which there is no road access. Since the site is designated for industrial use, it would require 23,302 gallons/day of sewage treatment capacity.

(5) CAPACITY AND POPULATION SURPLUS

Table 10 depicts the impact on surplus sewage treatment capacity of the development of additional uncommitted sites. The equivalent population surplus from existing and committed development was identified as 14,544 persons. The development of remaining lands within the former Town limits and north of Silver Creek which are not constrained by ownership, lack of access or other factors could decrease the equivalent population surplus by 1,048 persons to 13,496 persons. Additional lands north of the Creek are designated for urban use in the Official Plan although the likelihood of their being developed is limited. Assuming ultimate development of these constrained sites as visualized at present, the equivalent population surplus would be further reduced by 2,212 persons to 11,284 persons.

TABLE 10 EXISTING, COMMITTED AND UNCOMMITTED DEVELOPMENT

GEORGETOWN URBAN AREA

TOWN OF HALTON HILLS

FLOW SOURCE	SEWAGE TREATMENT ALLOCATION (gal/day)	EQUIVALENT POPULATION[1]
POTENTIALS:	5,000,000	40,000
Existing and Committed Development	3,182,043	25,456
SURPLUS:	1,817,957	14,544
Developable Residential	45,506	
Developable Commercial	3,204	
Developable Industrial	82,301	
TOTAL DEVELOPABLE:[2]	131,011	1,048
SURPLUS:	1,686,946	13,496
Constrained Residential	205,570	
Constrained Industrial	70,882	
TOTAL CONSTRAINED:[2]	276,452	2,212
SURPLUS:	1,410,494	11,284

[1] NOTE: Based on overall flow contribution rate of 125 gcd.

[2] NOTE: Lands north of Silver Creek only.

SECTION 5 BOUNDARY SUITABILITY

The Study Area covers approximately 5,360 acres of land. The former Town limits outline a land area of approximately 3,380 acres or 63% of the Study Area total, which includes approximately 380 acres of land south of Silver Creek. As indicated on Appendix B, a substantial portion of the urban area is developed or currently committed to some form of urban development. This, together with existing development in the rural area represents an equivalent population of 25,456 persons or, alternatively, an equivalent population surplus of 14,544 persons. Ultimate development of additional lands within the former Town limits and north of Silver Creek would lower the equivalent population surplus to 11,284 persons.

The likelihood of accommodating an equivalent population of 11,284 to 14,544 persons by urban infilling and in those portions of the Study Area not already accounted for would ultimately depend on permitted densities and physical constraints. The availability of developable land within the current Study Area boundaries, particularly in the rural portion, is limited by existing rural development as well as significant sensitive areas, woodlots, orchards and flood plains as indicated on Appendix E.[1]

If future development of those lands occurs at densities and use mixes comparable to those in place at present within the developed urban area north of Silver Creek and if no allowance is made for choice (no surplus allowance for land vacancy), then it would appear that the present Study Area boundaries enclose a sufficient amount of land to accommodate the entire equivalent population surplus of 11,284 persons. The appropriateness of current densities and mixes, the concept of an allowance for choice and the strategic locational merits of all such developable lands must be investigated in later phases of Stage II of this Study.

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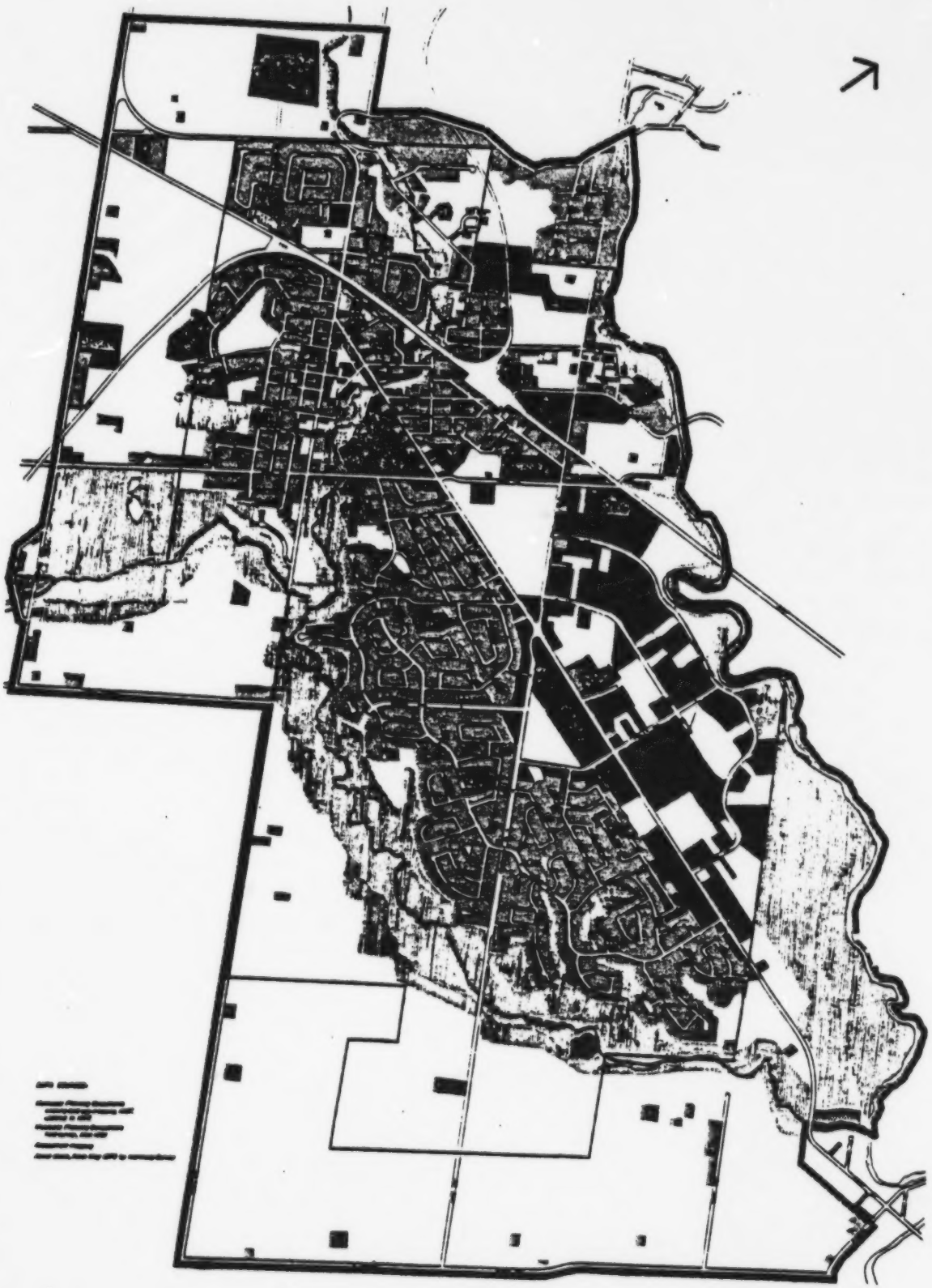
[1] NOTE: Floodlines indicated on the development constraints mapping are the subject of an ongoing study. These lines were preliminary when made available as they had yet to be reviewed by Conservation Authority officials.

REVISED: 12 Nov. 1981

APPENDICES

APPENDIX A
SERVICE AREAS
(DRAINAGE)
Georgetown Urban Area
Town of Halton Hills



APPENDIX B
1981 LAND USE
Georgetown Urban Area
Town of Halton Hills

APPENDIX C

INDUSTRIAL LOT COVERAGE

Industrial development in the Georgetown area on lots of 1 to 5 acres has tended toward a lot coverage of between 26% and 31%. Development on larger lots (5 to 10 acres) has tended toward approximately 15% lot coverage. More details are set out in Table C1.

Based on past experience in Georgetown, it seems unlikely that new industrial development, in the initial phase, will occur at the maximum lot coverage permitted by Zoning By-law No. 57-91. Instead, this Report assumes that development will be one storey and that lot coverage will average 40% for lots under 5 acres and average 15% for lots over 5 acres.

TABLE C1 INDUSTRIAL LOT COVERAGE (1981)

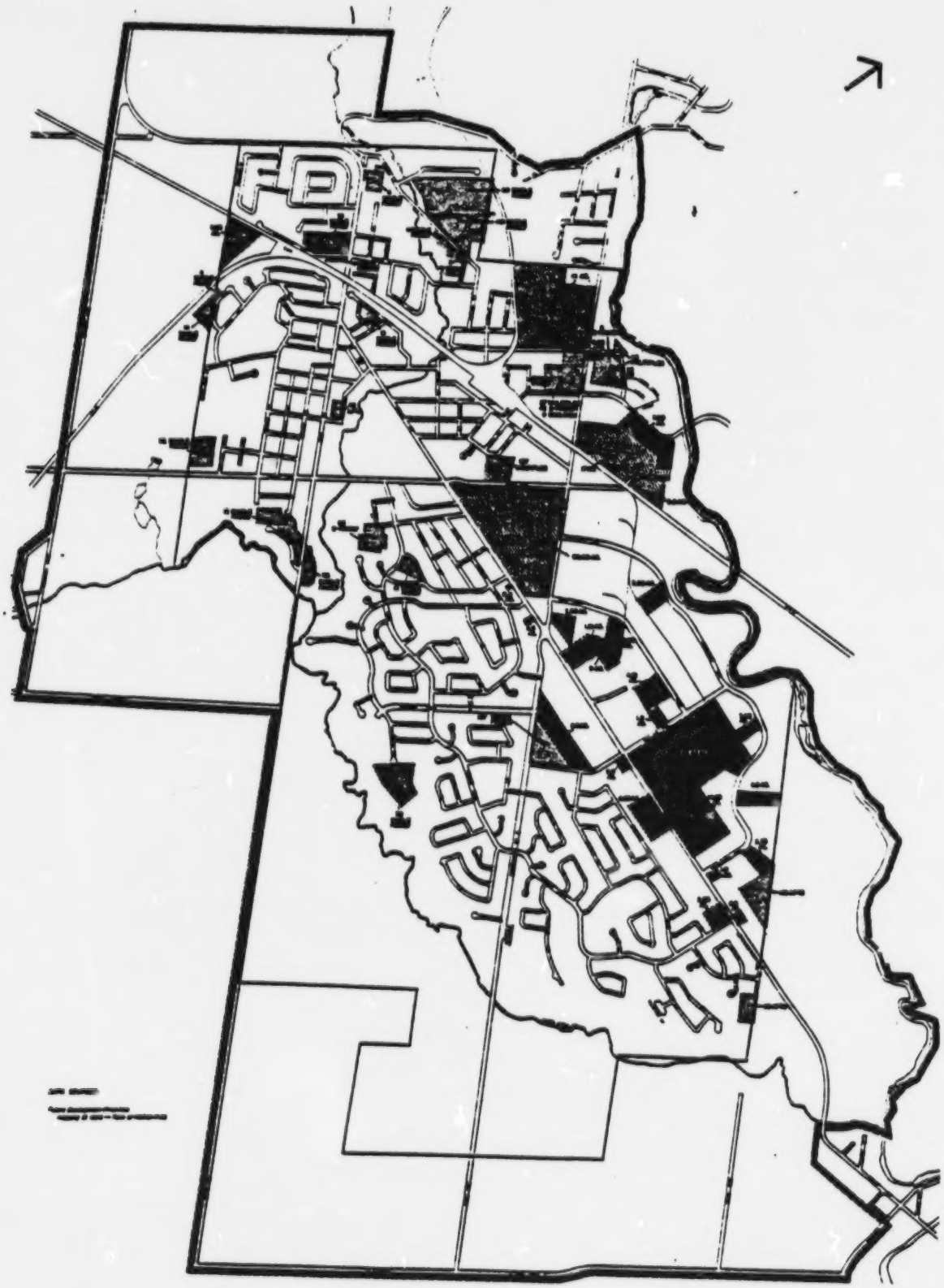
GEORGETOWN URBAN AREA

TOWN OF HALTON HILLS

INDUSTRIAL LOT SIZE (acres)	LOT COVERAGE[1] AVERAGE	MEDIAN
0-1	30.8%	27.9%
1-2	29.9	25.8
2-3	26.1	21.0
3-4	26.1	20.2
4-5	29.9	14.2
5-10	14.5	12.0

1 SOURCE: Estimates, Municipal Planning Consultants, 1981.

REVISED: 12 Nov. 1981



APPENDIX D
DEVELOPMENT STATUS
Georgetown Urban Area
Town of Halton Hills

APPENDIX E
DEVELOPMENT CONSTRAINTS
Georgetown Urban Area
Town of Halton Hills

APPENDIX E

TERMS OF REFERENCE

REGIONAL MUNICIPALITY OF HALTON

HALTON HILLS (GEORGETOWN) URBAN AREA REVIEW

SEWERAGE AND SEWAGE TREATMENT TERMS OF REFERENCE

Introduction

The present Georgetown sanitary sewage system consists of approximately 45 miles of sanitary sewers ranging in size from 8" to 30", 3 sewage pumping stations within the collection system, a main sewage pumping station at the plant site, and a plant with a design capacity of 3.0 mgd.

Serious infiltration problems exist in the collection system and it is desirable to confirm the plant's treatment capacity before the treatment plant reaches design capacity. Therefore, an assessment of the Georgetown sanitary sewage collection system is required. This assessment is to include the determination of existing sewage flows in the system and the location of point inlet infiltration water that could be readily eliminated. Utilizing information on population, commercial floor areas, industrial floor areas, and gross land area, determine the present residential, commercial and industrial sewage flows for average and maximum day, and peak hour. Conduct tests at the sewage treatment plant to determine the effluent quality obtainable when the plant is operated at 3.0 mgd average daily flow, and also for maximum day and peak hour following the current design basis for the plant. Determine the sewage flows possible in the present plant while still maintaining an effluent quality suitable for discharge to Rogers Creek and also the plant expansion possible. The current design was based on an average day sewage flow of 3.0 mgd with an effluent BOD of 10 mg/L. Report and plans by Proctor & Redfern Limited, dated April 1977 and February 1979 and by Philips Planning and Engineering Limited are available from the Region. These documents provide a complete description of the sewerage system. In addition, Simcoe Engineering are currently conducting an assessment of sewage plant sludge disposal for the Region, and Regional staff have reported on repairs carried out to reduce infiltration to the sanitary sewer system. Also the Ministry of the Environment has completed an assessment of the present plant discharge and its effect on Rogers Creek during 1979.

The following scope of work is a brief summary of the extent of the investigation to be completed. The study proposal should not include an allowance for laboratory testing. Instead, following the selection of the consultant, the consultant is to submit a complete testing schedule to the committee. The Ministry of the Environment staff will assess the current work load at the Ministry's laboratory and it is possible that this work could be carried out at no cost to the project. It is the consultant's responsibility, however, to deliver the samples to the lab and to arrange for the testing program.

Objectives of the Study

The sewerage and sewage treatment section of the study is required to demonstrate the population increases possible in the Georgetown urban area whilst consistently maintaining a level of performance at the pollution control plant, operating at plant design flow, which would produce an effluent quality within the assimilative capacity of the receiving stream, and in addition, maintain surface water quality goals, policies, and other objectives of the Ministry of the Environment.

The monitoring and testing of flow through the plant and in the receiving stream to take place over a time period of sufficient duration to ensure that the required level of performance can be achieved on a year-round basis.

The present equivalent population, i.e. with industrial and commercial development considered to be equivalent to 15 residents per acre, has been set to a maximum of 30,000 persons by the Ministry of the Environment pending the more detailed study envisaged in this report. The work of the study set out in greater detail below is required to indicate the following:

(a) Increased equivalent population, if any, which can be permitted with the existing pollution control plant.

(b) Further increases in equivalent population which can be permitted using additional waste water purification techniques permitted under current M.O.E. regulations.

Scope of Work

1. A meeting will be arranged by the consultant with the committee to review, amend as required, and finalize the complete work program, including the scheduling of all work items, monitor and testing techniques, etc. The consultants will provide a schedule of sampling and testing to be carried out at the plant for review. The Ministry will then determine its capability in carrying out the laboratory analysis. In addition, it is known that the treatment plant does not include the necessary measurement/control features to enable monitoring and regulating the sewage flows and the return sludge flows. Temporary equipment will have to be installed by the consultant for this work.

2. Direct all sewage flows to plant B and determine the treatment efficiency under different flow conditions. Determine the treatment achieved in the primary, aeration, final settling and filtration plant process, under the different flow conditions and the actual organic loadings.

The Region will assist the consultant in the operation of the plant under the test conditions, and in the collection of samples testing. However, the consultant is expected to provide expert staff as required for the control of particular plant functions that would require more than the normal time and knowledge afforded by the operational personnel. Further, it should be noted that regional operating staff are in attendance at the plant from 7:30 a.m. to 4:00 p.m. Monday through Friday, and from 7:30 a.m. to 11:30 a.m. on weekends and holidays.

The objective is to determine the capability of the plant to continue to perform under design flow condition. Also, it is a prerequisite to the establishment of the additional treatment units required, should it be concluded that the plant can be further expanded.

In addition, the plant test program shall be sufficient to demonstrate the rate of build-up of nitrifying bacteria in the Spring months and that the system continues to perform adequately during the low Summer and Fall stream flow conditions.

3. During the full scale plant tests, the consultant shall carry out tests as appropriate to monitor the upstream and downstream water quality in Rogers Creek and in the Credit River below the confluence.

4. Review the work being concluded by Simcoe Engineering on sludge disposal for the Region and determine the requirements applicable for the Georgetown plant at 3.0 mgd, and for any expansion beyond 3.0 mgd.

5. Determine the extent of by-passing at the main sewage pumping station on the plant site by installing an appropriate flow measuring device in the by-pass sewer. Also, the sewage flows in the Rogers Creek and the Flamingo Court trunk sewers are to be monitored. The Rogers Creek trunk services areas 5 to 19 (as described in the P & R Report) comprising approximately 853 acres, the older section of Georgetown, and the Flamingo Court sewer serves areas 1 to 4 which include the lands south-east of Sargent Road and Mountainview Road with an area of approximately 665 acres. Area 4 includes the existing industrial development.

6. Monitor the sewage flows at the Armstrong Avenue sewage pumping station which serves area 4, the John Street station which serves area 16, and the Moore Park station which serves area 10.

7. Monitor sewage flows in manhole 1 (at the old S.T.P. site) on the Rogers Creek trunk sewer and compare to the sewage flows received at the plant pumping station to determine if there is a significant amount of infiltration along this section of the trunk.. Check tops of three manholes, upstream from

present plant to confirm that overflows cannot occur at these locations. A similar check is required for the Flamingo Court trunk sewer across the Rogers Creek valley.

It is expected that with the completion of items 5, 6 and 7, the extent of by-passing at these locations will be known and that the consultant will determine those where realistic corrective measures should be instituted.

8. Determine the population served by the Georgetown sewage system utilizing the most recent data sources available, such as assessment information. (Note that the Halton Hills Planning Department may now have current figures) Also determine the population served by the John Street and Moore Park sewage pumping station and the commercial floor area in these service areas.

9. Determine the existing total commercial floor area and industrial floor area plus the developed industrial land area. (Note that the Halton Hills Planning Department may now have current figures.)

10. Determine the water consumption, from Regional Public Works Department staff, for commercial and industrial uses for average day, and maximum day per month, and the total water consumption for Georgetown for average day, maximum day per month, and peak hour for 1975 to present.

Examine the water consumption charts for 1979 to determine the correlation if any, with the total sewage flows for 1979 on a daily and peak hour per day basis for wat and dry periods. Note that S.T.P. flow meter records are not accurate prior to September, 1978. The M.O.E. has stream flow data for the station located on Rogers Creek downstream from the plant.

11. Determine the changes to industrial development since September 1976, including water consumption for major users for 1976, 1977, 1978 and 1979.

12. Previous consultant reports have confirmed the existence of significant infiltration flows into the sanitary collection system. This item includes an examination of previous reported conditions, discussions with Region Operations and Maintenance staff, and a comparison of past and present conditions. The Region has carried out most of the recommended repairs to the system, and it is necessary that these be documented.

    (a) Check for the existence of overflows between manhole #9 and the storm sewer in Area 1 on the Flamingo Court trunk. If this overflow continues to exist, it should be plugged for the duration of the monitoring program in this study.

(b) Check the condition of manhole 274 top in Area 10 to see that it has been sealed against the entry of surface drainage water.

(c) Check the infiltration into manhole 791 on Sinclair Avenue in area 4.

(d) Confirm if the catchbasin on Chipper Court is connected to the sanitary sewer in Area 5.

(e) Confirm if the catchbasin is still connected to manhole 101 on Pauline Street in area 6.

(f) Confirm if significant infiltration exists between manhole 76 and manhole 77 in area 6.

(g) Confirm the location of the top of manhole 72 and check if it is still subject to flooding and infiltration.

(h) Confirm the presence of infiltration for the cluster of manholes at the intersection of Hwy. 7 and Maple Avenue.

(i) Confirm if surface drainage water continues to flow into the system via the connection to the Domtar sewage lagoon in area 16.

(j) Confirm the condition of the manhole north of the CNR on the Rogers Creek trunk.

(k) Confirm the condition of the brick work and cover for manhole 2 in area 19.

(l) Confirm the condition of the Rogers Creek trunk sanitary sewer between Ewing Street and CNR embankment. Infiltration was previously excessive in this section.

(m) Confirm the condition of the Princess Anne Drive sanitary sewer and the discharge point of the hospital footing drain sump pump.

(n) Confirm the drainage problem at manhole 87.

(o) Confirm the extent of infiltration from the abandoned stub sewer at manhole 22 on Princess Anne Drive.

(p) Confirm the continuation of the high sewage flows in the Mill Street service area (area 9), Park Street (area 8), John Street (area 14), and John Street (area 16).

(q) Inspect and record the extent of repairs to existing low manhole tops, bases and benching since September, 1976.

(r) Confirm the existence of an overflow between the sanitary and storm sewers at the intersection of Delrex Blvd. and Weber Drive.

(s) Confirm the flooding/infiltration condition at manhole 17, 51, 52, 53 on the Rogers Creek trunk.

(t) Review the situation with the Meadow Glen Mushroom Company, th: water quantity and quality discharged to the sanitary sewers, and the resultant impact if the water were discharged evenly throughout the day.

13. Determine the current per capita residential and institutional sewage flows for average day, maximum day and peak hour. Also project the time during the day when peak hour is most likely to occur due to domestic sewage flows. Determine the per capita average daily sewage flows for areas 10 and 16.

14. Determine the current commercial and industrial sewage flows on a floor area basis and industrial gross developed land area as appropriate for average and maximum day. Also, project the time during the day when peak hour flows are most likely to occur for commercial and industrial development.

15. Determine the residential per capita sewage flow, based on the system total sewage flow for existing conditions.

16. Identify locations of points of infiltration where corrective measures could be readily undertaken and recommend the corrections that should be carried out.

17. Based on the findings in items 12 and 16, project the wet and dry weather infiltration flows that could be eliminated if corrective measures were carried out. It should be noted that "credit" for the elimination of flows would only be considered after the necessary repairs have been completed and inspected after a period of time.

18. Project the resultant system total flows if the identified corrective measures as determined in item 16 were carried out and the resultant overall residential per capita sewage flows based on thc total system use (relate total system sewage flows to actual serviced population).

19. In completing items 8 to 18 inclusive, it is expected that the consultant will have sufficient data to detail the current sewage flow generation rates for different land uses. Further, that recommendations will be presented concerning additional measures that, if implemented, would reduce the sewage flows to the plant. The consultant will recommend sewage flow generation rates that would be appropriate for additional development, having regard to potential change in lifestyles, increased densities, and land uses that could result in an increase in flows over the current rates.

20. Determine the capacity of the sewage treatment plant to treat flows in excess of its design. (Compare the test results for the conditions of average day to maximum day to peak hour ratio versus the design conditions.) This assessment is to consider both the sewage flows and appropriate strength parameters, taking into consideration the expected changes in sewage strengths through elimination of infiltration with and new development. Determine the capacity of the sewage treatment plant units to treat sewage flow greater than 3.0 mgd and produce an effluent quality that does not exceed the assimilative capacity of Rogers Creek. The critical effluent parameters are known to include: nitrogen in all phases; phosphorous, total and soluble, BOD and residual chlorine. The Ministry's "Blue Book" on Water Management in Ontario, November 1978, shall apply. It should be noted that in this assessment, the full capacity of the creek will be utilized in the determination of the assimilation capacity with no reserve capacity being required. Only those parameters that are significant to the Georgetown plant are to be tested.

21. Review practical alternative treatment modifications that if incorporated into the present plant would achieve an increase in the treatment capacity within the assimilative capacity of the creek. Recommend the most appropriate treatment plant expansion program and tabulate the design parameters for the recommended additional works. Also tabulate the recommended revised design parameters for the existing plant treatment units.

22. Assess the pumping station and treatment plant to determine if mechanical or electrical standby equipment should be provided.

23. Prepare a conceptual site layout plan for the sewage treatment site showing existing and recommended works, having regard to the existing property limits of the S.T.P. site.

24. Prepare estimates of probable costs for capital works required to expand the treatment works to the ultimate capacity possible with discharge to Rogers Creek, and also the repairs necessary to the sanitary sewage collection system to reduce the current high levels of infiltration.

25. Prepare and submit 50 copies of the sewerage and sewage treatment section of the report with plans of all projects findings, assessments, recommendations, and estimates of probable costs. 25 copies of a summary of the report will be prepared, suitable for insertion into Public Works Committee and Regional Council agendas.

26. It is contemplated that regular project review meetings would be held with the consultant.

27. Investigate and allow for all subsidies available in the preparation of this report and work resulting from its implementation.

COSTS

Separate budget amounts should be itemized for on the following basis:

(a) Work at the sewage treatment plant site, including the main sewage pumping station.

(b) Sanitary sewage collection system infiltration investigation and analysis, including residential and institutional, commercial and industrial sewage flow calculation.

(c) Existing population and water use considerations.

APPENDIX 'F'

CORRESPONDENCE

Table of Contents

DECRIPTION	PAGE NO.
Letter Dated December 30, 1981 Trewin to Huggins re: MOE requirements	F-1
Letter dated August 20, 1981 Trewin to Huggins re: June 12, 1981 Meeting	F-3
Letter dated June 24, 1981 Huggins to Trewin re: Summary of Meeting of June 12, 1981	F-4
Letter dated January 7, 1981 Griffen to Palmer re: 7Q20 Flows in Rogers Creek and sampling program	F-7

Ontario

Ministry of the Environment

Central Region

Suite 700
150 Ferrand Drive
Don Mills, Ontario
M3C 3C3
(416) 424-3000

1981 12 30

RECEIVED
JAN 11 1982
GORE & STORRIE LTD.

Mr. Norm Huggins
Gore and Storrie Limited
1670 Bayview Avenue
Toronto, Ontario
M4G 3C2

Dear Mr. Huggins:

Re: Georgetown Sewage Treatment Plant

This is to confirm the following items from our meeting of December 18, 1981 concerning the possible expansion of the Georgetown Sewage Treatment Plant to 5.0 MIGD:

1. The annual low flow (7Q20), which normally occurs in the summer, in the Credit River, Georgetown Branch, upstream of the Georgetown sewage treatment plant discharge (Station 14 of the MOE 1979 Survey) is 144 L/sec. The winter (December-February) low flow (7Q20) at this location is 170 L/sec. This summer flow was previously provided in a letter from Giffen to Palmer, January 7, 1981. It is our understanding, that a copy of a memo, Shaw to Giffen which specified the winter flow was also forwarded to Palmer.

2. It is recognized that ammonia concentrations upstream from the Georgetown sewage treatment plant discharge during the winter are on occasion, sufficiently high to exceed MOE's un-ionized ammonia objective. This is a reflection of the character and uses of the stream above the plant's discharge. Recognizing the lack of flow for effluent dilution and assimilation and its quality, the un-ionized ammonia concentration in the plant's final effluent will have to be less than or equal to the Provincial Water Quality Objective for this parameter.

3. It is expected that to meet the chlorine objective of 0.002 mg/L downstream of the plant's discharge, de-chlorination of the final effluent will be required.

4. The BOD_5 in the final effluent will be expected to average 5 mg/L or less.

5. The plant will be expected to produce an effluent phosphorus concentration of 0.3 mg/L or less.

If there are any questions concerning the above, would you please contact Mr. R. Shaw of this office.

Yours truly,

G.R. Trewin, P.Eng.
Manager
Municipal and Private Abatement Section

RS:cra:MSC3-C

Ontario

F-3

Ministry of the Environment
Central Region

Suite 700
150 Ferrand Drive
Don Mills, Ontario
M3C 3C3
(416) 424-3000

1981 08 20

RECEIVED
AUG 24 1981
GORE & STORRIE LTD.

Mr. M. Huggins
Project Coordinator
Gore and Storrie Limited
1670 Bayview Avenue
Toronto, Ontario
M4G 3C3

Dear Sir:

Re: Town of Halton Hills
Urban Review (Georgetown)
Our File No. MP32-02

Your correspondance of June 24, 1981 reported on the June 12, 1981 meeting at which a progress report was presented by yourself and others on the study being undertaken for the region with respect to the capacity of the Georgetown Sewage Treatment Plant and its receiving waters. I believe the reporting of the meeting, as indicated in your letter, to be fair and we look forward to receiving the final material which will be prepared on the completion of the survey work. One qualification on the reporting could be that with respect to the 40,000 person estimate presented on the bottom of page 2. Additional works may have to be installed depending on your studies to ensure that a liberal design is available with respect to effecting a satisfactory reduction of ammonia, and other critical criteria if identified, in the effluent.

Yours truly,

G.R. Trewin, P.Eng
Manager
Municipal and Private Abatement Section

GRT:cra:MS3-E

TR/CCP. X

F-4

F-4

603.01

28 June 1981

Mr. George Trewin, P.Eng.
Manager
Ontario Ministry of the Environment
Central Region
Municipal & Private Abatement Section
Suite 700
150 Ferrand Avenue
Don Mills, Ontario
M3C 3C3

Gentleman:

Re: Regional Municipality of Halton, Town of Halton Hills
Urban Review (Georgetown)

Thank you for the opportunity you gave us on June 12, 1981 to present the findings of the above study. The exchange of ideas and thoughts on future action to be taken.were appreciated and will be considered in the remainder of our study.

To summarize our findings we note the following:

1. The study has been set up into 5 basic tasks:
   a) Data collection and review
   b) Determination of MOE limits
   c) Sampling and flow measurement at the water pollution control plant
   d) Sampling and flow measurements in the collection system.
   e) Determination of stream loadings and mixing zones in Roger's Creek.
2. We noted that all aspects of the study were under control and proceeding without major difficulty.

.../2

3. The work in the collection system has been carefully monitored. We have studied a broad range of conditions under which the dry weather flows have been determined. We believe that there are high instantaous loads at the plant as the result of low manhole covers and poor manhole construction in the main trunk line coming to the plant through the river valley.

4. The work at the water pollution control plant has been underway since last October and we have been able to determine that plant A and B perform very well under normal dry weather flows. In February all of the flow was directed to plant B in order to examine the impact of higher than design flows on the nitrification process. The results under load conditions of 2.2 million gallons plus, indicates that nitrification is not feasible at this loading rate. We believe that nitrification can be obtained somewhere between 1.5 and 2.2 mgd. We have requested the steering group to authorize us to reactivate plant A, after which we will study the ability of plant B to re-establish nitrification under various loadings.

   Monitoring of the plant operations will continue until this fall at which time we believe the study will be reasonably complete.

5. The stream assimilation study has been underway since January with monitoring periods set for a one month period per season, one in January, one in April, one in July and one in October. The results of the first two months of monitoring are recorded on the sheets distributed at the meeting and represent the preliminary results of sampling and testing.

   We discussed the affects of pH and temperature on the unionized ammonia in the stream. With the exception of a short period during the April sampling when the pH rose to 8.8, which we believe may be attributed to local geology and a quarry operation, we believe that the pH will remain within the normal limits of a stream and the temperature ranges will be according to the season.

It was generally agreed that the meeting was a good meeting and that the study has been reasonably intense and thorough to date.

When asked whether or not the town could proceed with the planning portion of the study, assuming the population equivalent of 40,000 person plus or minus, it was indicated that based upon the findings to date and our estimate of the requirements to upgrade the existing facility in certain of the unit processes that there is a distinct possibility that the service pppulation may be increased.

.../3

F-6

F-6

You noted the following:

a) that the Ministry would not allow a submission of flow data based on a dry year.

b) the relationship of the stream flows to the 7Q20 flow for Roger's Creek must also be evaluated and established as part of the final report.

On behalf of the Town, the Region and ourselves, we would like to thank you for the opportunity you have given us in presenting this information and your comments on same as they relate to the extension of the study into the planning stage. We will endeavor to keep you and the Ministry informed as the project progresses in order that you may be continually aware of our activities.

Yours very truly,

N. D. Huggins, P. Eng.
Project Co-ordinator

NDH:mp

cc: H.N. Cripps, P. Eng.
A. Mohammed
Mayor Pameroy
Carl Reichert
D. Barber

603.01

F-7

Ontario

Ministry of the Environment

Central Region

Suite 700
150 Ferrand Drive
Don Mills, Ontario
M3C 3C3
(416) 424-3000

1981 01 07

Gore & Storrie Limited
Consulting Engineers
1670 Bayview Avenue
Toronto, Ontario
M4G 3C2

RECEIVED
JAN 12 1981
GORE & STORRIE LTD.

Attention: Dr. M.D. Palmer, Ph D.,
Water Resources Division

Dear Sir:

Re: Town of Halton Hills (Georgetown)
Urban Area Review Study
Stream Water Quality

This follows our discussions concerning stream water quality as it relates to the Halton Hills (Georgetown) Urban Area Review Study.

As requested by Halton Region, we are forwarding a Draft Copy of a report prepared by our Technical Support Section on the water quality survey done in 1979 on the Black Creek - Credit River West Branch. We anticipate some discussion with Halton Region before the report is finalized for general release.

In our discussions about your study, questions have been raised regarding the acceptable plant effluent loadings and the use of mixing zones in Rogers Creek downstream of the plant discharge. As indicated in the terms of reference for this study, the "Blue Book" is applicable to this project and presumably all five policies on surface water quality management would apply. If acceptable loading limits for the plant effluent are required at this stage of the study to provide guidance in the conduct of your investigations, these may be approximated using the water quality parameters specified in the "Blue Book" for the 7Q20 flow in Rogers Creek of 5.1 cfs and the plant design flows being chosen. A reasonable estimation of stream characteristics above the sewage treatment plant outfall is given on page 16 of our draft report.

We also have discussed item 3 of the study terms of reference which requires the consultant during the full-scale plant tests to carry out tests as appropriate to monitor upstream and downstream water quality in Rogers Creek and in the Credit River below the confluence.

In order to characterize water quality for the period of your study, we recommend four sampling runs, with each run covering four consecutive weeks for the winter (mid-January to mid-February), spring (April), summer (mid-July to mid-August) and fall (October) for a total of 16 sample dates. The sampling days should coincide with plant sampling days. Four stream sampling stations are recommended initially as follows:

1. within 30-100 metres upstream of the plant outfall;

2. 30-100 meters downstream of the plant outfall;

3. one concession downstream of the plant site; and

4. at the confluence of Rogers Creek with the Credit River.

Depending on the early stream monitoring test results, it may be necessary to locate the fourth monitoring station downstream of the confluence and establish a fifth station in the Credit River upstream of the confluence with Rogers Creek.

We hope this information will aid you in setting up of a program for assessing receiving stream water quality as part of the overall study. If there are any question or if additional clarification is required, please contact the writer.

A copy of this letter has been forwarded to Mr. H.H. Cripps, Region of Halton for his information.

Yours truly,

A.V. Giffen, P. Eng.
District Officer
Municipal & Private Abatement Section
ns/W-K

POOR

QUALITY

ORIGINAL